“STRUCTURAL ANALYSIS AND DESIGN OF A MULTISTORY BUILDING AND STUDYING THE EFFECT OF MASONRY INFILLS ”

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

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&

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to

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HIMACHAL PRADESH, INDIA

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CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"STRUCTURAL ANALYSIS AND DESIGN OF A MULTISTORY BUILDING AND STUDYING THE EFFECT OF MASONRY INFILLS "** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering and submitted to the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by Aatish (Enrolment no. 131636) and Rahul Sharma (Enrolment no. 131638) during a period from July 2016 to June 2017 under the supervision of Prof. Dr. Ashok Kumar Gupta (Professor and Head) and Mr. Chandra Pal Gautam (Assistant Professor) , Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of our knowledge.

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The topic **"STRUCTURAL ANALYSIS AND DESIGN OF A MULTISTOREY BUILDING ADN STUDYING THE EFFECTS OF MASONARY INFILLS**" was very helpful to us in giving the necessary background information and inspiration in choosing this topic for the project. Our sincere thanks to **Prof. Dr. Ashok Kumar Gupta ,** Project Guide; **Mr. Abhilash Shukla;** Project Coordinator; **Mr. Chandra Pal Gautam,** Faculty Advisor and **Mr. Bibhas Paul**, Faculty advisor for having supported the work related to this project. Their contributions and technical support in preparing this report are greatly acknowledged.

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Aatish (131636) **Rahul Sharma (131638)**

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ABSTRACT

As per the data released by the Central Statistics Office (CSO), the predictable growth in economic actions during the 21st century is 4.8% in construction and 2.8% in transportation. With mounting in globalization there is less land accessible for construction, thus the need of construction of off shore and on shore structures has amplified with time. But off shore and on shore structures are affected by wind, sea waves in addition to earthquake forces. Thus our project aims at design and construction of a structure that is along the coast line that is more prone to seismic actions.

The consequence of masonry infill panel on the response of RC frame subjected to seismic action is widely documented and has been subject of several experimental investigations, while several attempts to model it analytically have been reported. Infill behaves like compression between column and beam and compression forces are transferred from one node to another. In this project the effect of masonry walls on high rise building is studied. Static analysis on high rise building with different arrangement is carried out. For the analysis G+15 R.C.C framed building covering G+10 and steel building for rest G+5 is modelled.

Steel is more flexible material than concrete hence for high rise buildings steel is used for design. Concrete is quite good in compression, economical and easily available therefore must be used for designing structures. But when it comes to tall buildings , concrete provides major disadvantage that is poor tensile strength. Hence reliance on concrete cannot be laid in such situations. Therefore it is the steel that is to be used. Being light in weight, strong and more ductile it provides more liability against seismic hazards. So the main aim is to provide steel con concrete design that could be both economical and safe.

CHAPTER 1: INTRODUCTION

1.1 GENERAL INTRODUCTION

It is a well known fact that among the natural hazards, earthquakes have the potential for causing the greatest damages to engineered structures. Since earthquake forces are random in nature & unpredictable, the engineering tools needs to be sharpened for analyzing structures under the action of these forces. India has a number of the world's greatest earthquakes in the last century. In fact, more than fifty percent area in the country is considered prone to damaging earthquakes. Thus it is important to build structures that are resistant to earthquake.

1.2 AIM OF PROJECT

This project aims for relearning of concept of structural design with the help of computer aids.

- To find the effect shown by different natural forces on a structure and to find the most critical case that could arise (or to determine the most sever case effecting the structure).
- Design various structural elements of a multi-storey building manually and thereby comparing results with software.
- Designing and understanding of earthquake resistance structure design concept.
- Design of foundations.
- Using infill materials to make the structure earthquake resistant. Improving the seismic vulnerability of the structure.
- Studying the effects of using masonary infill at different locations in a structure.

1.3 IS CODES

- IS:456-2000: Design Code For Rcc Structures
- IS:800-2007: Structural Steel Design
- IS:875(Part-1): Code For Dead Loads
- IS:875(Part-2): Code For Imposed Loads
- IS:875(Part-3): Code For Wind Loads
- IS:1893.1.2002: Code For Earthquake Load
- IS:1904–1986: Code of practice for design and construction of foundations in soils : general requirements

1.4 ABOUT THE PROJECT

1. Utility of building: Residential building(hotel)
2. No of storeys: G+15
3. No.of staircases: 2 each floor
4. Shape of the building: Rectangular(35.60m×21.60m)
5. Construction Material: R.C.C framed(first 10 floors) and steel framed(next 5 floors) structure
6. Type of walls : Brick wall, RC Wall
7. Project site

   Chowpati beach , Porbandar ,Gujarat.

   Earthquake zone -3

   Average wind velocity-50m/s

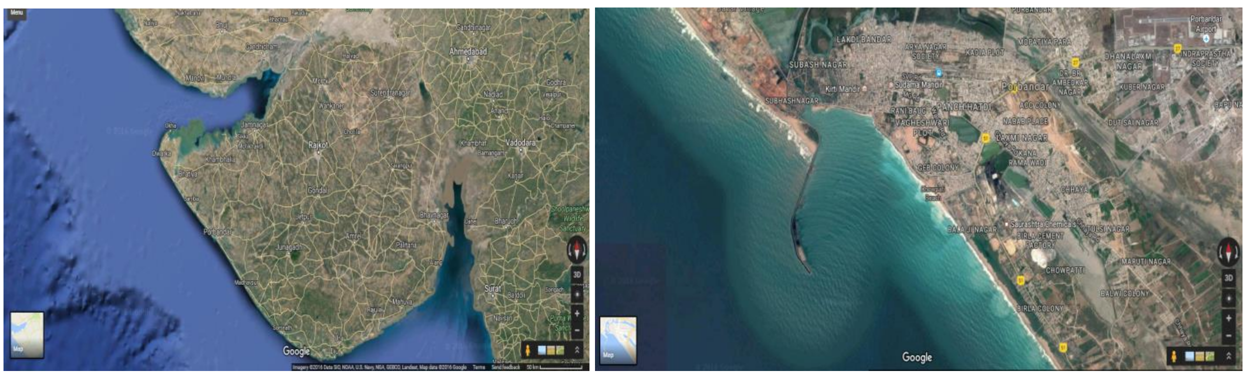


Figure 1: Site of project

1.5 SOFTWARES USED

- STAAD PRO V8i
- Auto-Cad: 2013

CHAPTER 2 : LITERARY REVIEW

1. "A Study on Earthquake Resistant Construction Techniques" (Mohammad Adil Dar ,Prof (Dr) A.R. Dar ,Asim Qureshi ,Jayalakshmi Raju)

Apart from the modern techniques which are well documented in the codes of practice, there are some other old traditional earthquake resistant techniques which have proved to be effective for resisting earthquake loading and are also cost effective with easy constructability.

In addition to the main earthquake design code 1893 the BIS(Bureau of Indian Standards) has published other relevant earthquake design codes for earthquake resistant construction Masonry structures (IS:13828-1993)

- Horizontal bands should be provided at plinth ,lintel and roof levels as per code
- Providing vertical reinforcement at important locations such as corners, internal and external wall junctions as per code.
- Grade of mortar should be as per codes specified for different earthquake zones.
- Irregular shapes should be avoided both in plan and vertical configuration.
- Quality assurance and proper workmanship must be ensured at all cost without any compromise.

In RCC framed structures (IS:13920)

- In RCC framed structures the spacing of lateral ties should be kept closer as per the code
- The hook in the ties should be at 135 degree instead of 90 degree for better anchoragement.
- The arrangement of lateral ties in the columns should be as per code and must be continued through the joint as well.
- Whenever laps are to be provided, the lateral ties (stirrups for beams) should be at closer spacing as per code.

2. "Behavior And Strength Of RC Column-To-Steel Beam Connections Subjected To Seismic Loading" (Gustavo J Parra-Montesinos And James K Wight)

This paper tells about the hybrid structures particularly RCS frames and the seismic behavior of reinforced concrete column-to-steel beam (RCS) connections is studied. In RCS frames, the advantages of both reinforced concrete and steel structures are combined to form a cost and time effective type of construction. RC columns are more cost effective in terms of axial strength and stiffness than the steel columns [Sheikh et al. 1987]. Also, they offer superior damping properties to the structure, especially in tall buildings. On the other hand, steel floor systems are lighter and require little or no formwork, reducing the weight of the building and thereby increasing the speed of the construction.

The influence of different joint details on the seismic response of the connections is discussed. These joint details are two-part U-shaped stirrups passing through drilled holes in the steel beam web, steel cover plates or band plates surrounding the joint region, dowel bars attached to the steel beam flanges, and steel fiber concrete or engineered cementitious composite (ECC) material in the connection. Experimental results indicate that RCS frames are suitable for use in high seismic risk zones. In addition, good agreement was found between experimental results and the shear strengths predicted by the proposed model.

3. "**Analytical Review Of Soft Storey"** (Ghalimath.A.G, Hatti M.A)

Multi-storey buildings in metropolitan cities require open taller first storey for parking of vehicle and/or for retail shopping, large space for meeting room or a banking hall owing to lack of horizontal space and high cost. Due to this functional requirement, the first storey has lesser strength and stiffness as compared to upper stories, which are stiffened by masonry infill walls. This characheristics of building construction creates weak or soft storey problems in multi storey buildings. Increased flexibility of first storey results in extreme deflections, which in turn, leads to concentration of forces at the second storey connections accompanied by large plastic deformation. In addition, most of the energy developed during the earthquake is dissipated by the column of the soft stories. In this process the plastic hinges are formed at the ends of column, which transform the soft stories into a mechanism. In such cases the collapse is unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design. It has been observed from the survey that the damages are due to collapse and buckling of columns especially where parking places are not covered appropriately. On the contrary, the damage is reduced considerably where the parking places are covered adequately. It is recognized that this type of failure results from the combination of several other unfavorable reasons, such as torsion, exceesive mass on upper floors, P-Δ effects and lack of ductility in the bottom storey.

In case of buildings with a flexible storey, a special arrangement needs to be made to increase the lateral strength and stiffness of the soft storey. Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly those in the soft storey, and the members are designed accordingly.

This phenomena of soft story may arise due to many different reasons such as change in load carrying and slab system between stories. The abrupt changes which take place in the amount of the infill walls between stories is also one of the frequent reasons of the soft storey behavior. Since infill walls are not regarded as a part of load carrying system, generally civil engineers do not consider its effects on the structural behavior.

4. "Beneficial Influence Of Masonry Infill Walls On Seismic Performance Of Rc Frame Buildings" (C. V. R Murty And Sudhir K Jain)

Masonry infills in reinforced concrete buildings cause several undesirable effects under seismic loading like short-column effect, soft-storey effect, torsion and out-of-plane collapse. Hence, seismic codes tend to discourage such constructions in high seismic regions. However, in several moderate earthquakes, such buildings have shown excellent performance even though many such buildings were not designed and detailed for earthquake forces. This paper presents some experimental results on cyclic tests of RC frames with masonry infills. It is seen that the masonry infills contribute significant lateral stiffness, strength, overall ductility and energy dissipation capacity. With suitable arrangements to provide reinforcement in the masonry that is well anchored into the frame columns, it should be possible to also improve the out-of-plane response of such infills. Considering that such masonry infill RC frames are the most common type of structures used for multistorey constructions in the developing countries, there is need to develop robust seismic design procedures for such buildings.

Masonry infill wall panels increase strength, stiffness, overall ductility and energy dissipation of the building. More importantly, they help in drastically reducing the deformation and ductility demand on RC frame members. The reinforcement in the infills does not contribute significantly towards stiffness and strength; in fact it may lead to reduction in stiffness and strength due to increased mortar thickness in the layers containing the reinforcement. However, the reinforcement helps in improving the post-cracking behaviour of the masonry and in preventing out-of-plane collapse. Most multistorey building constructions in the developing countries consist of RC frames with URM infills. Often the RC frame is not even formally designed for seismic loading even in severe seismic zones. This situation is not likely to change significantly in the near future. Such buildings are commonly used as residential or office buildings which typically have a fairly large number of infills placed more or less uniformly and have small to moderate panel size. It should be possible to develop suitable detailing schemes for anchoring masonry reinforcement into the frames and thereby improve the out-of-plane behaviour of the infills. In such situations, the infills could be relied upon to ensure good seismic performance.

5. "Seismic Design of Masonry and Reinforced Concrete Infilled Frames: A Comprehensive Overview" (Marina L. Moretti)

The most frequently encountered Infilled Frames are the unreinforced brick masonry panels built in the space between columns and beams in a reinforced concrete building. The brick infills have proved to increase the seismic response of bare reinforced concrete frames in terms of strength, stiffness and energy dissipation capacity (Abrams, 1994; Bertero and Brokken, 1983; Govidan *et al*., 1986; Manos *et al*., 1995). The presence of a regular pattern of infills in layout and in height of the structure prevents energy dissipation from taking place in the frames (Negro and Verzeletti, 1996). Masonry infills continue to govern the overall response of buildings with reinforced concrete moment-resisting frames even after cracking of the masonry walls (Murti and Nagar, 1996). In modeling of a new concrete building, the contribution of the masonry infills to the lateral resistance is generally ignored. The reinforced concrete structural elements are designed to resist the entire seismic demand. However, in case the masonry infills may have negative effects on the global response of a building, then the infills should be included in the structural model (CEN, 2004; Fardis, 2009). Typical examples are the irregular distribution of infills (Negro and Taylor, 1996; Negro and Colombo, 1997) and the case of partial-height infill walls that do not extend to the full height of the column which may result in columns experiencing non-ductile shear failure rather than responding in a predominantly flexural manner (Moretti and Tassios, 2006; 2007; Yuen and Kuang, 2015).

The present paper reviews the in-plane behavior of reinforced concrete frames with masonry and RC infills and reviews the different types of available models, with emphasis on the engineering model most often used, that of the diagonal strut. Code provisions for the design of infilled frames are presented and their applicability to masonry and RC infilled frames (i.e., frames with RC infills) is discussed.

Infill walls increase significantly the stiffness of a RC frame building. However, their presence may also be detrimental to the overall earthquake performance of the structure, especially when the building is not designed according to modern principles. Modeling of infilled frames is complex because of their highly non-linear behavior when subjected to horizontal loading.

The equivalent diagonal strut model is a practical engineering tool for the design of infilled frames. The type of strut model adopted, however, can alter significantly the results. Therefore, the strut model characteristics should be selected according to the objective of the analysis. In case of RC infills, the connection between infill and frame strongly affects seismic behavior and should be appositely modeled.

CHAPTRE-3 :SOFTWARE WORK

3.1 AUTOCAD PLAN

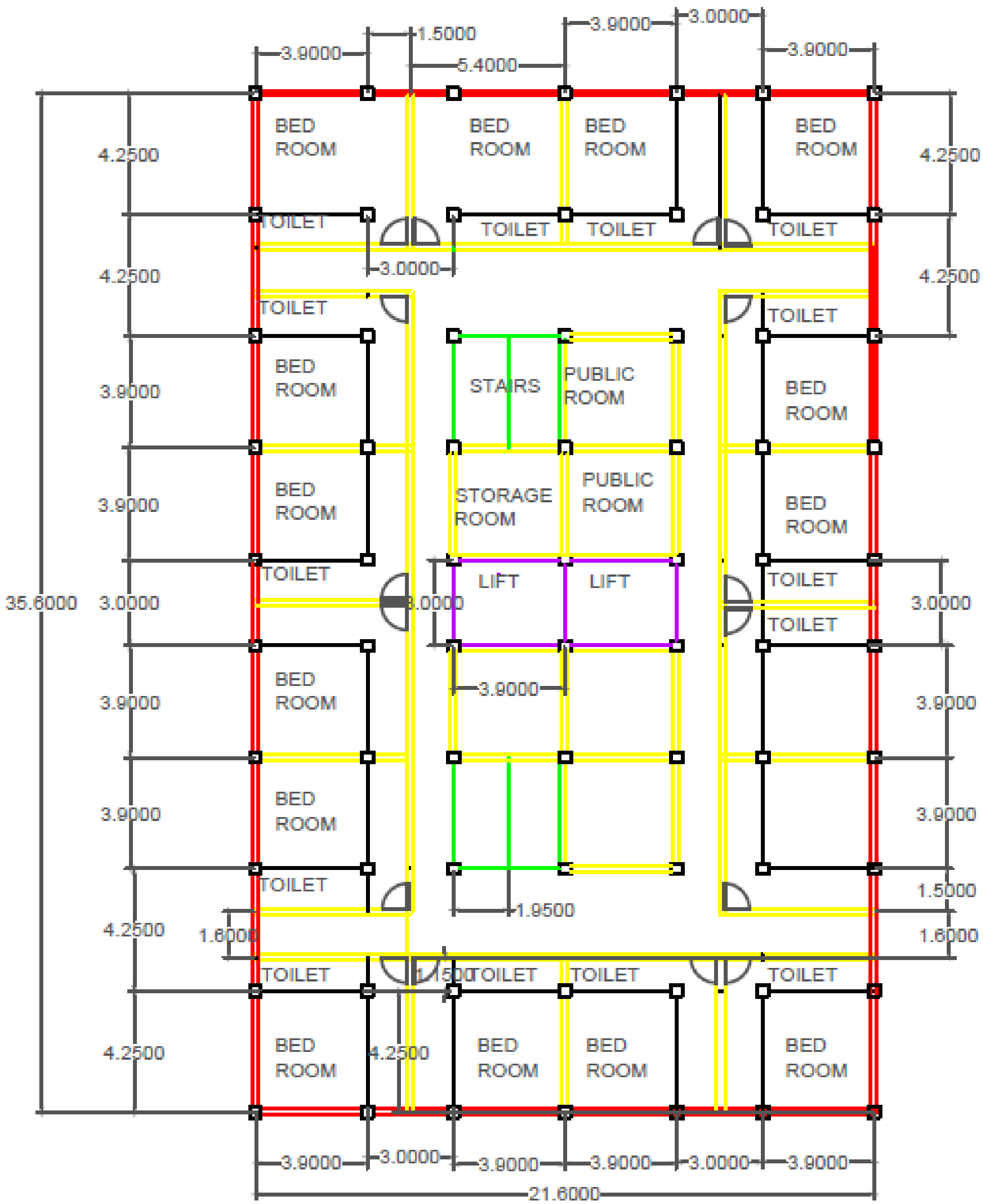


Figure 2: Auto-cad Plan

3.2 STAAD MODEL

A)TOP VIEW

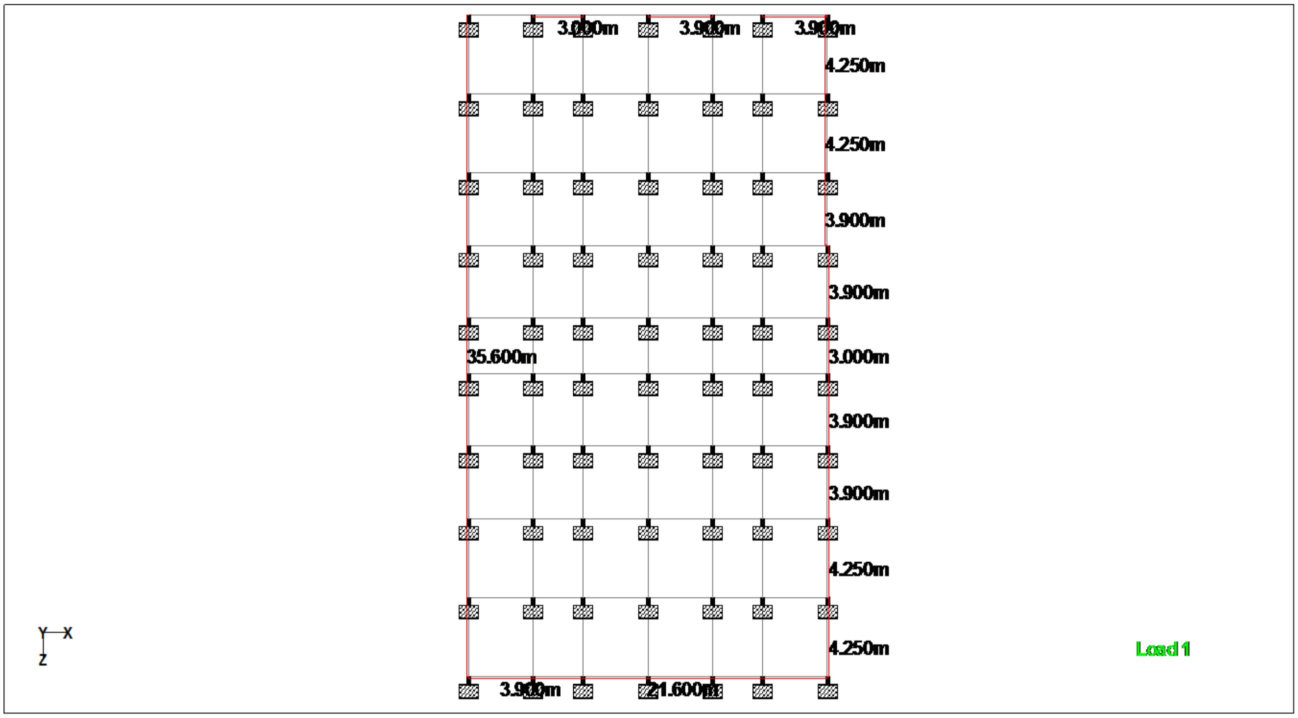


Figure 3: STAAD Model top view

B) FRONT VIEW

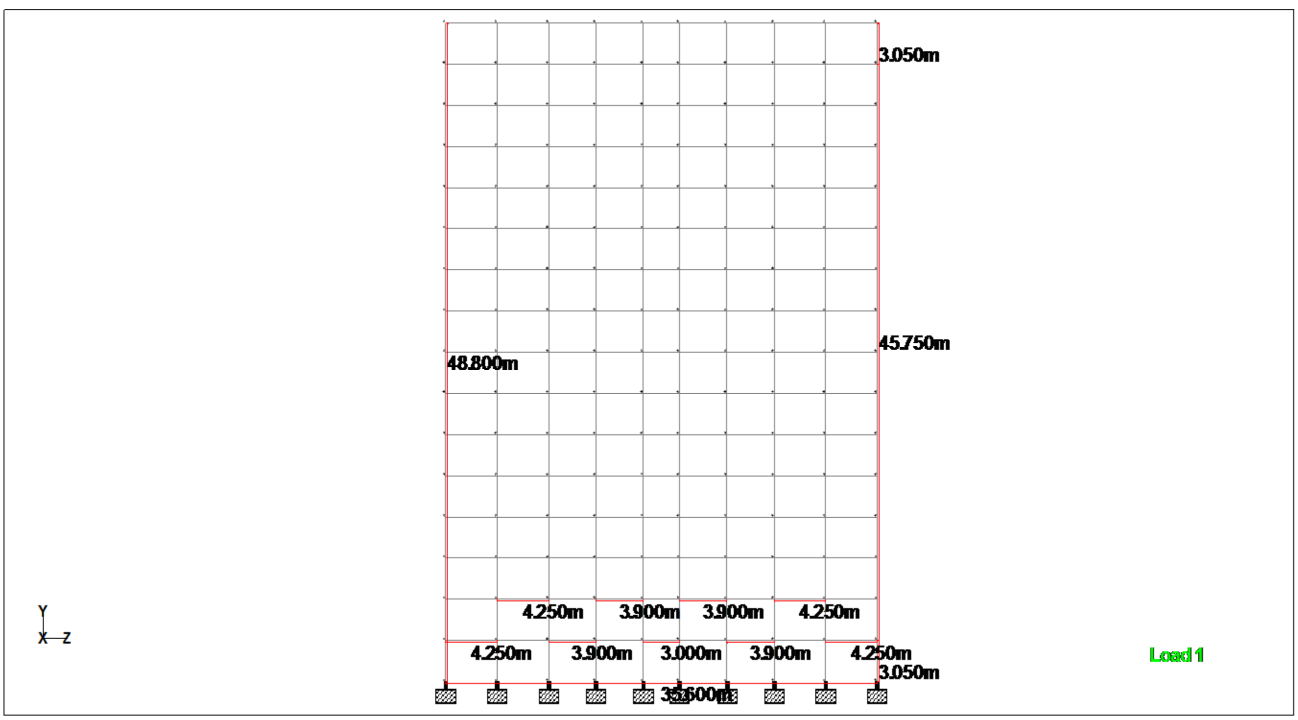


Figure 4: STAAD model front view

C) SIDE VIEW

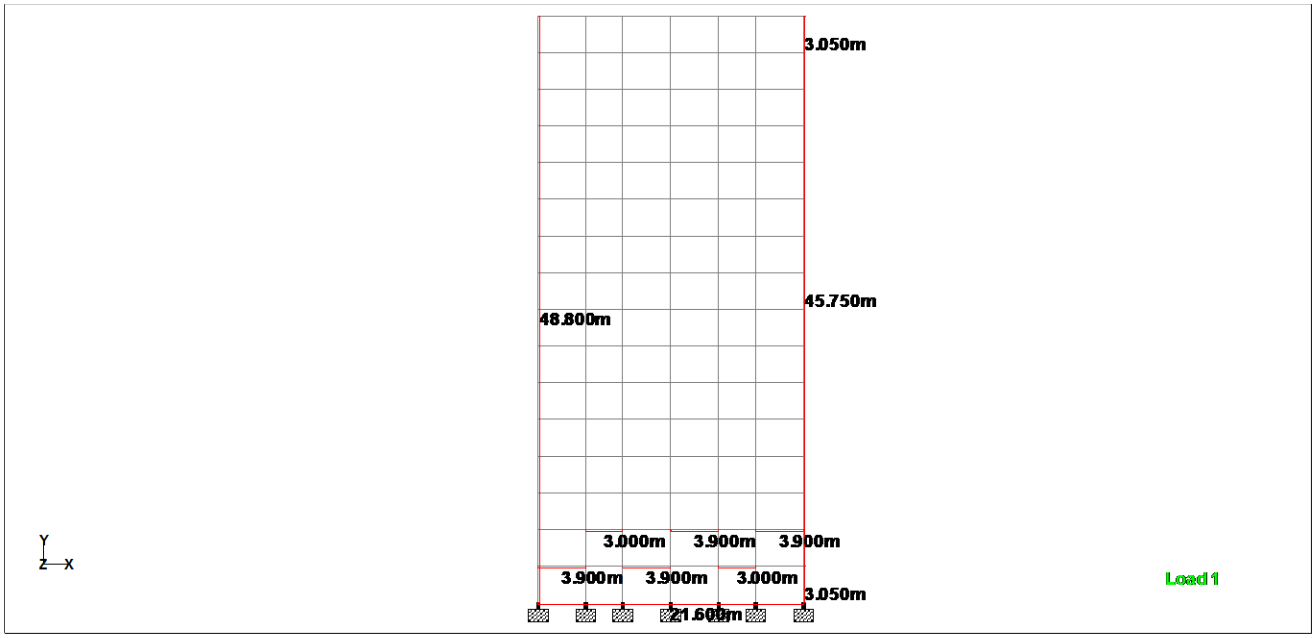


Figure 5: STAAD model side view

D)3-D MODEL

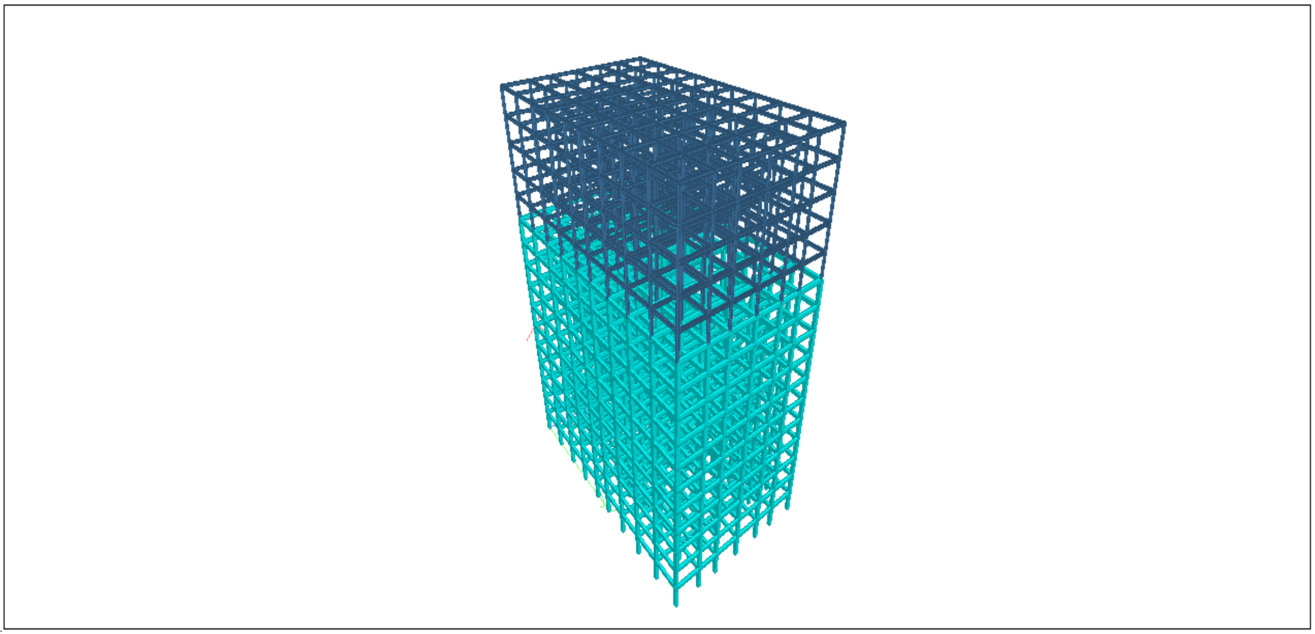


Figure 6: STAAD 3-D model

CHAPTER 4- LOADING

According to the specifications given in IS:875-1987 :Part-1,2,3 and IS:1893:Part-1-2002 the following loads will be acting on the structure.

1. Dead load- IS:875-1987 (Part-1)
2. Live load- IS:875-1987 (Part-2)
3. Wind load- IS:875-1987 (Part-3)
4. Earthquake load IS:1893 (Part-1):2002

4.1 DEAD LOAD

- Dead load refers to the self weight of the structure covering the weight of each and every element of the structure like the beam, column, slab and the walls.
- The unit weight of various building material are given in IS:875 (part 1).

VARIOUS DEAD LOADS APPLIED:

1. Finishing load=1KN/m^2
2. Wall Load: Engineering masonry brick *(Table-1, Point-36)*

   Unit weight= 23.55KN/m^3

   Thickness of wall=200mm

   Wall load=4.71KN/m
3. Slab Weight:

   Unit weight of concrete= 25KN/m^3

   Slab thickness=200mm

   Slab weight=5KN/m^2

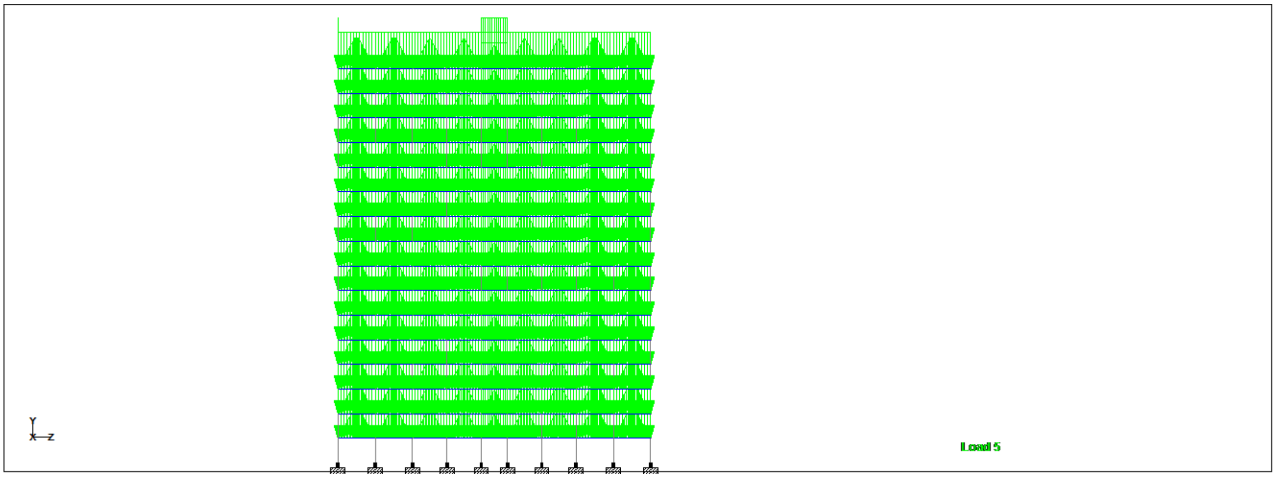


Figure 7: Dead Load Front View

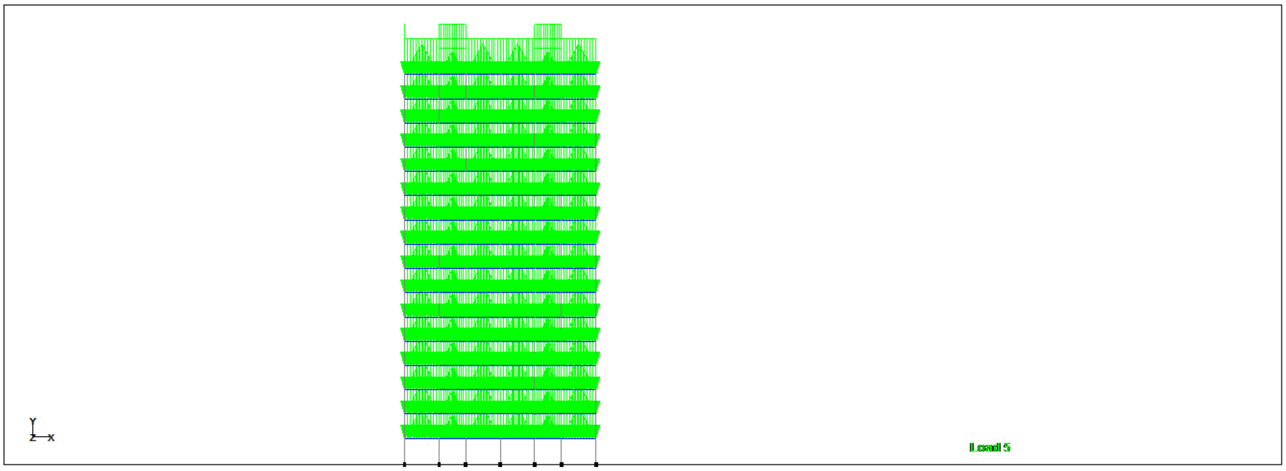


Figure 8:Dead Load Side View

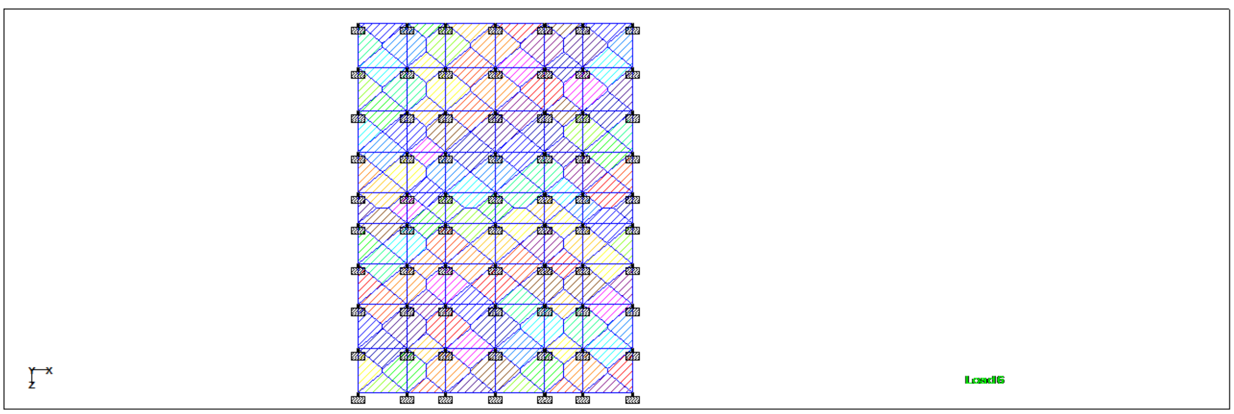


Figure 9: Floor Load Top View

4.2 IMPOSED LOAD

- The use of the term ‘live load’ has been modified to ‘imposed load’ to cover not only the physical contribution due to persons but also due to nature of occupancy , the furniture and other equipments which are a part of the character of the occupancy. (Clause-0.3.2)
- The IS 875 (part 2) gives specification of various live loads.
- *Residential Buildings - These shall include* . any building in which sleeping accommodation is provided for normal residential purposes with or without cooking or dining or both facilities. It includes one multi-family dwellings, apartment houses flats), lodging or rooming houses, restaurants , hostels, dormitories and residential hotels. (Clause 2.2.1 of IS:875-1987 (Part-2)).

<u>HOTELS:</u>

- Clause 2.2.1 of IS:875-1987 (Part-2), says that hotels comes under residential buildings.
- As per IS:875-1987 (Part-2), from Table-1 and Table-2 the following are the loads coming from different occupancies and from the roof.

Table 1 : Live load(KN/m^2)

PUBLIC ROOM (Table-1)	3
STORAGE ROOM (Table-1)	5
STAIRS (Table-1)	3
BATHS AND TOILET (Table-1)	2
ROOF(ACCESSIBLE) (Table-2)	1.5
ROOFS(IF NOT ACCESSIBLE)	0.75

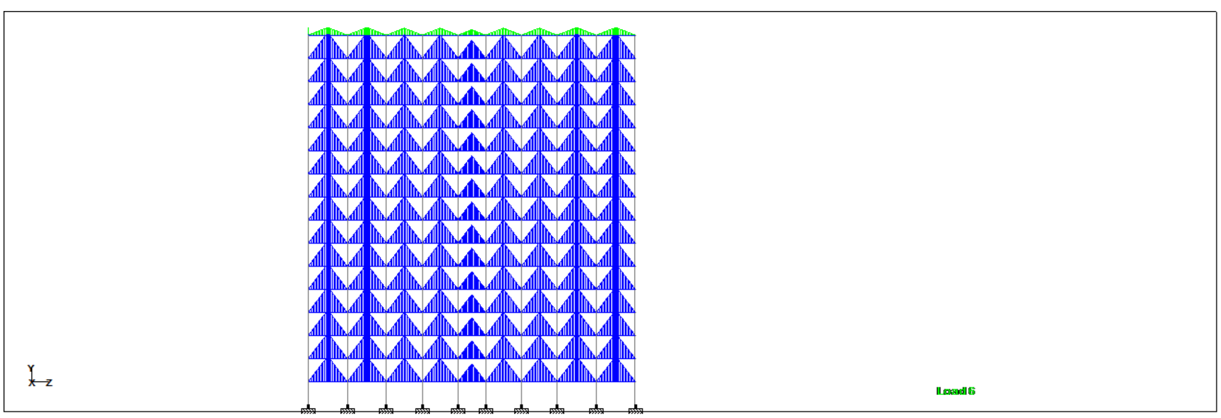


Figure 10: Live Load Front View

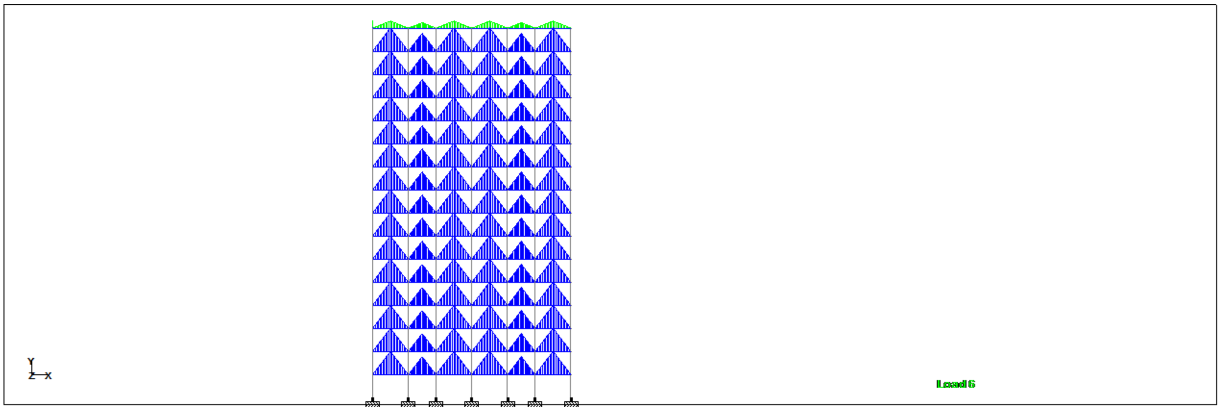


Figure 11: Live Load Side View

4.3 WIND LOAD

The part-3 deals with wind loads to be considered when designing buildings, structures and components thereof. In its second revision in 1987, the important modifications were made from those covered in the 1964 version of IS: 875.

Wind causes a random time-dependent load, which can be seen as a mean plus a fluctuating component.

All structures will experience dynamic oscillations due to the fluctuating component (gustiness) of wind. In short rigid structures these oscillations are insignificant, and therefore can be satisfactorily treated as having an equivalent static pressure. A structure may be deemed to be short and rigid if its natural time period is less than one second. The more flexible systems such as tall buildings undergo a dynamic response to the gustiness of wind.

DYNAMIC EFFECTS OF WIND (Clause-8, IS:875, Part-3)

In general, the following guidelines may be used for examining the problems of wind induced oscillations:

a) Buildings and closed structures with a height to minimum lateral dimension ratio of more than about 5.0, or

b) Buildings and closed structures with natural frequency in the first mode less than 1.0Hz.

(a) Height of the building,H= 48.80m

Least lateral dimension,B= 21.60m

H/B= 48.80/21.60 = 2.25

H/B < 5

(b) For moment resistant frames without bracings or shear walls for resisting the lateral loads

$$T = 0.1 \times n$$

where

n = number of storeys including basement storeys;

and

For all others $T = 0.09H/d^{(1/2)}$

H = total height of the main structure of the building in meters, and

d = maximum base dimension of building in meters in a direction parallel to the applied wind force.

The structure contains shear walls hence the second formulae is used.

$T = 0.09H/d^{(1/2)} = 0.09 \times 48.80/35.60^{0.50} = 0.7361$sec.
f=1/T = 1/0.7361 = 1.3585Hz
f > 1Hz

Since both the conditions (a) and (b) are satisfied therefore there is no need to carry out the dynamic effect of wind on the structure.

Table 2 : Wind load calculation

pd	z	Uz	K1	K2	K3	Ub	Pz	Ka	Kd	C
1152.48	3.05	49	1	0.98	1	50	1440.6	0.8	1	0.6
1152.48	6.1	49	1	0.98	1	50	1440.6	0.8	1	0.6
1152.48	9.15	49	1	0.98	1	50	1440.6	0.8	1	0.6
1194.2469	12.2	49.88	1	0.998	1	50	1492.809	0.8	1	0.6
1230.1875	15.25	50.625	1	1.013	1	50	1537.734	0.8	1	0.6
1297.4208	18.3	51.99	1	1.04	1	50	1621.776	0.8	1	0.6
1340.0647	21.35	52.8375	1	1.057	1	50	1675.081	0.8	1	0.6
1379.0208	24.4	53.6	1	1.072	1	50	1723.776	0.8	1	0.6
1418.5351	27.45	54.3625	1	1.087	1	50	1773.169	0.8	1	0.6
1455.3019	30.5	55.0625	1	1.101	1	50	1819.127	0.8	1	0.6
1475.5245	33.55	55.44375	1	1.109	1	50	1844.406	0.8	1	0.6
1495.8867	36.6	55.825	1	1.117	1	50	1869.858	0.8	1	0.6
1516.3884	39.65	56.20625	1	1.124	1	50	1895.486	0.8	1	0.6
1537.0297	42.7	56.5875	1	1.132	1	50	1921.287	0.8	1	0.6
1557.8105	45.75	56.96875	1	1.139	1	50	1947.263	0.8	1	0.6
1578.7308	48.8	57.35	1	1.147	1	50	1973.414	0.8	1	0.6

Where:-
Vb = Basic Wind Speed
Vz = Design Wind Speed
k1 = probability factor (risk coefficient) (see5.3.1),
k2 = terrain roughness and height factor (see 5.3.2),
k3 = topography factor (see 5.3.3)
k4 = importance factor for the cyclonic region (see 5.3.4)
pz = wind pressure in N/m2 at height z
pd =Design Wind Pressure
Kd = Wind directionality factor
Ka = Area averaging factor
Kc = Combination factor

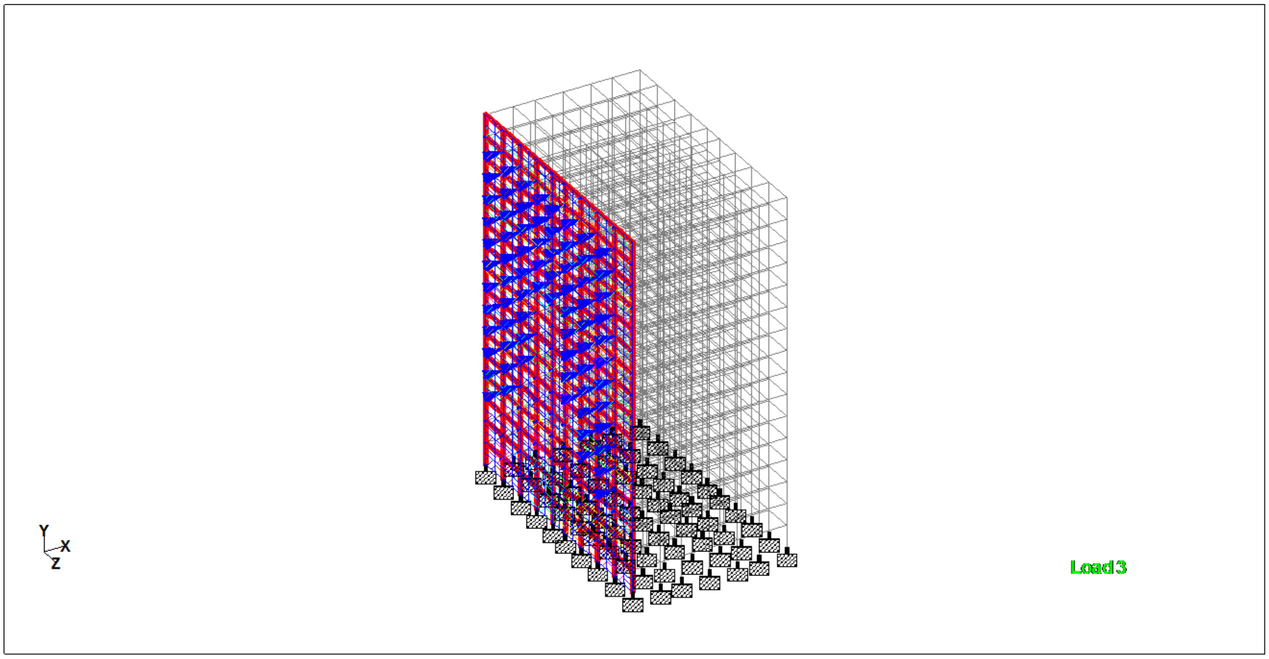


Figure 12: Wind Load

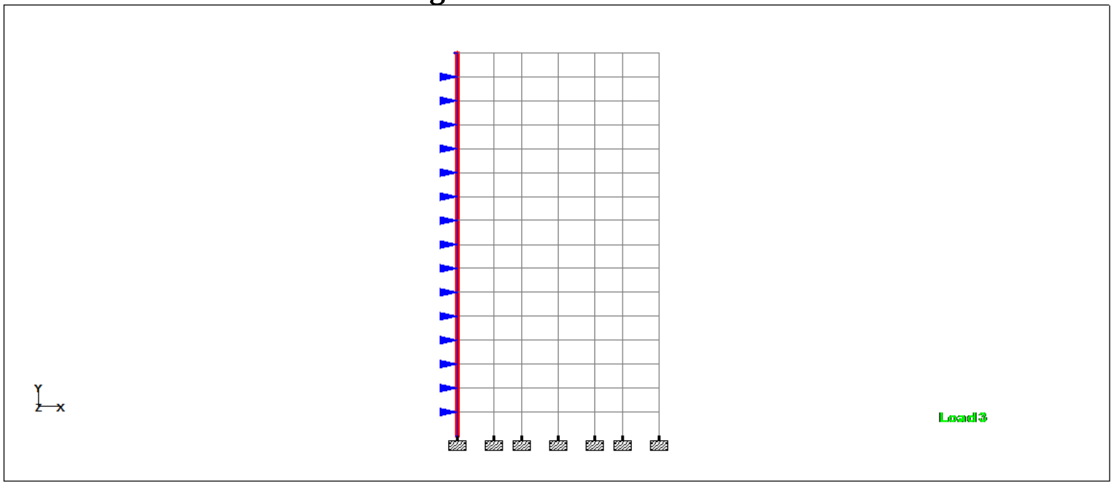


Figure 13: Wind Load From Side view

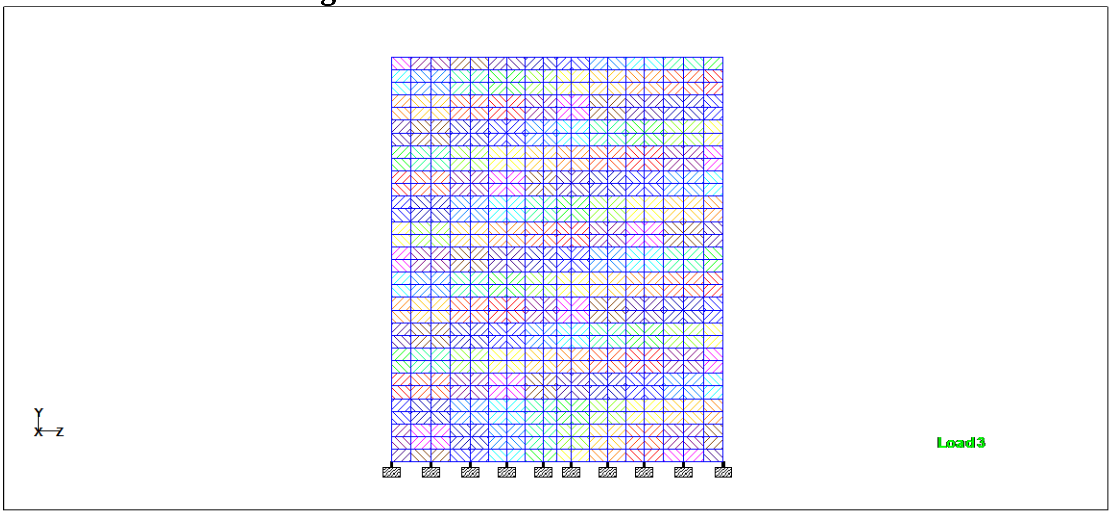


Figure 14: Wind Load From Front View

4.4EARTHQUAKE LOAD

Seismic load calculations will be done following IS1893(Part l)-2000.The seismic weights are calculated in a manner similar to gravity load. The weight of columns and walls in any story shall be equally distributed to the floors below and above thestory. Following reduced live loads are used for analysis .Zero of roof and 50% on floors.

Z= Zone Factor

I =Importance Factor

R= Response Reduction Factor

Fundamental Natural Period (T_a)

It is the first (longest) modal time period of vibration.

T_a =0.075$h^{.075}$ for RC frame building

T_a =0.085$h^{.075}$ for steel frame building.

Design Seismic Base Shear (V_b)

It is the total design lateral force at the base of a structure.

$V_b = A_h \times W$

A_h=design horizontal acceleration spectrum

W=sesmic weight of building

$A_h = Z \times I \times S_a/(2 \times R \times g)$

SESMIC LOAD CALCULATION

sesmic zone 3. Zone factor(Z) = 0.16

Importance facto(I)e= 1.5

response reduction factor(R)=5 for steel frame and 3 for OMRF

floor area= 768.96 m^2

live load acting on floors= 5 KN/m^2

live load= 1.5 KN/m2 roof

% considered= 50%

Dead load= 6 KN/m^2

sesmic weight contribution from each floor= 6536.16 KN

sesmic weight contribution from each roof= 5190.48 KN

total sesmic weight of structure= 103232.88 KN

Fundamental naturl period of vibration=

X-direction= $0.09 \times h/d^{0.5}$ = 0.941134953 sec.

Z-direction= $0.09 \times h/d^{0.5}$ = 0.733084066 sec.

Sa/g (X-direction) = 1.445063745			h = 48.6 m
Sa/g=Z direction	1.855176049		L = 35.6 m
$A_h = Z \times I \times (Sa/g)/(2 \times R)$	0.05780255	OMRF(X)	b = 21.6 m
$A_h = Z \times I \times (Sa/g)/(2 \times R)$	0.03468153	steel(X)	
$A_h = Z \times \times I(Sa/g)/(2 \times R)$	0.074207042	OMRF(Z)	
$A_h = Z \times I \times (Sa/g)/(2 \times R)$	0.044524225	steel(Z)	

design base shear(KN)V_b=

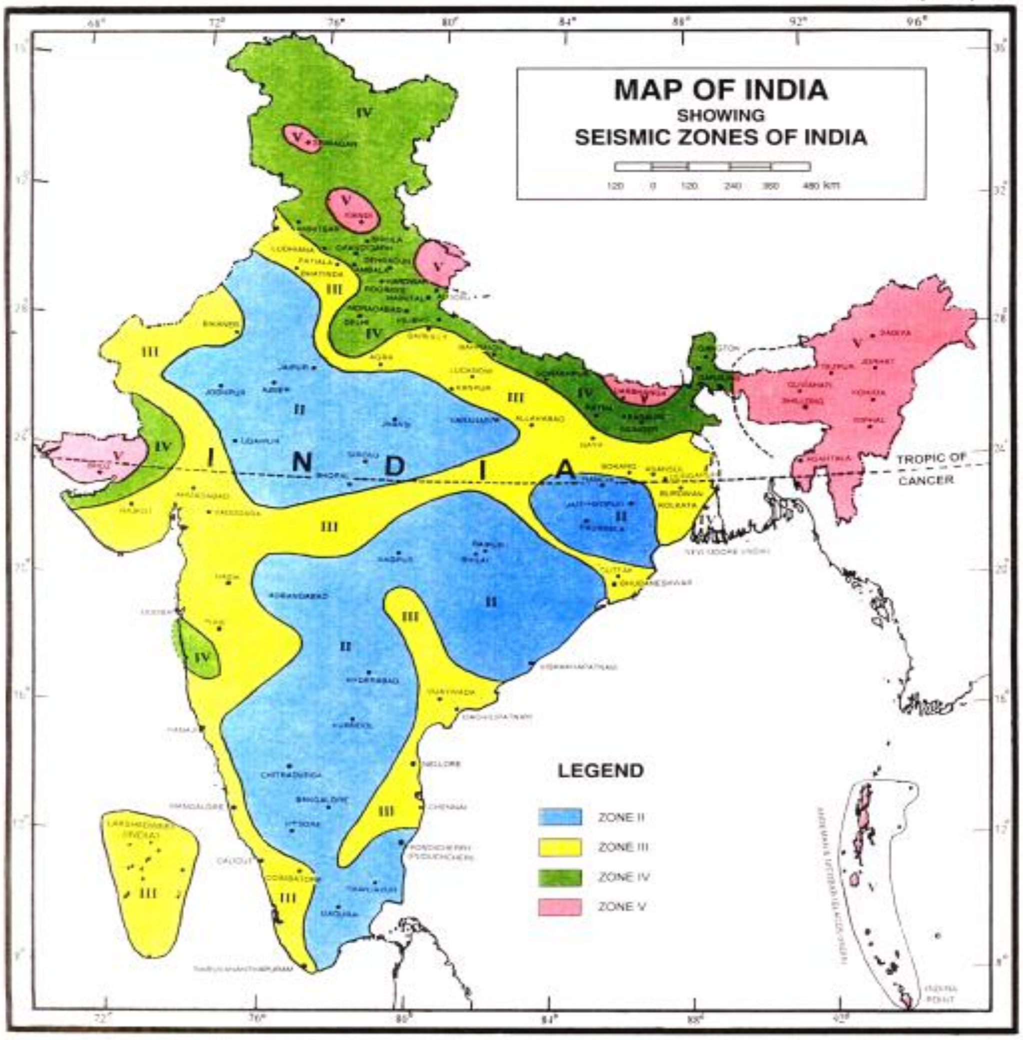
Xdirection=	5091.501068
Z direction=	6536.480391

Table 3: seismic load calculation

Lateral load distribution with height X direction Z direction

story level	W (KN)	h(m)	W×h²	W×h²/∑(W×h²)	Q=V_b(W×h²/∑(W×h²)) (KN)	Q=V_b×(W×h^2/∑(W×h^2))² (KN)
roof	5190.48	48.8	12360816.69	0.1409	717.1595897	920.6910755
15	6536.16	45.75	13680591.39	0.1559	793.7313167	1018.994029
14	6536.16	42.7	11917315.17	0.1358	691.4281692	887.6570208
13	6536.16	39.65	10275644.2	0.1171	596.1804112	765.3777373
12	6536.16	36.6	8755578.49	0.0998	507.9880427	652.1561785
11	6536.16	33.55	7357118.036	0.0838	426.8510636	547.9923444
10	6536.16	30.5	6080262.84	0.0693	352.7694741	452.8862351
9	6536.16	27.45	4925012.9	0.0561	285.743274	366.8378504
8	6536.16	24.4	3891368.218	0.0443	225.7724634	289.8471905
7	6536.16	21.35	2979328.792	0.034	172.8570423	221.9142552
6	6536.16	18.3	2188894.622	0.0249	126.9970107	163.0390446
5	6536.16	15.25	1520065.71	0.0173	88.19236852	113.2215588
4	6536.16	12.2	972842.0544	0.0111	56.44311585	72.46179761
3	6536.16	9.15	547223.6556	0.0062	31.74925267	40.75976116
2	6536.16	6.1	243210.5136	0.0028	14.11077896	18.1154494
1	6536.16	3.05	60802.6284	0.0007	3.527694741	4.528862351
∑			87756075.91		5091.501068	6536.480391

IS 1893 (Part 1) : 2002



NOTE : Towns falling at the boundary of zones demarcation line between two zones shall be considered in High Zone.

- Based upon Survey of India map with the permission of the Surveyor General of India.
- The responsibility for the correctness of internal details rests with the publisher.
- The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base line.
- The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.
- The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.
- The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

FIG. 1

5

Figure 15: Seismic zone distribution in India

CHAPTER-5 MATERIALS

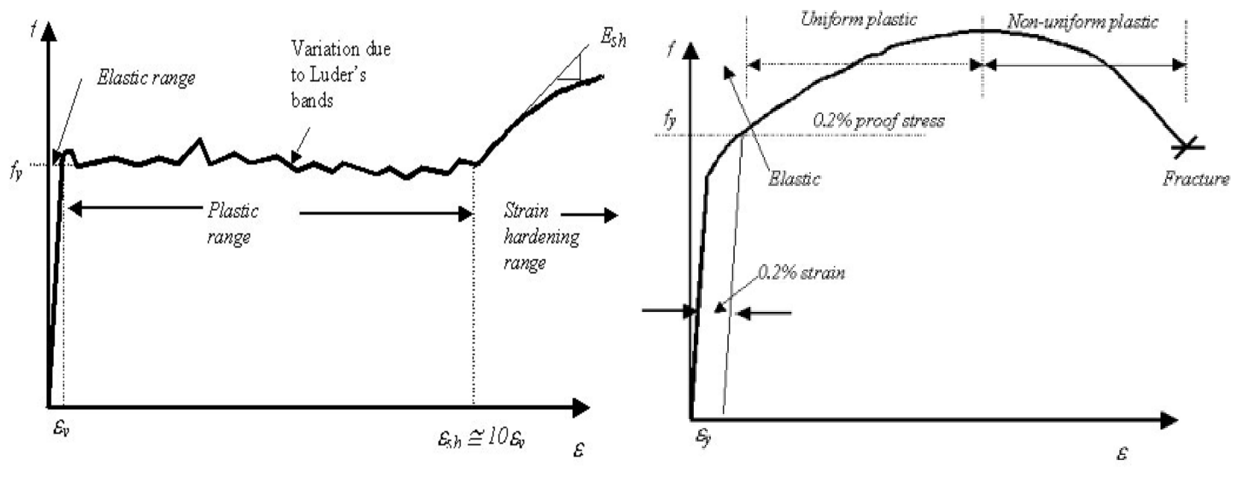
5.1 STEEL

ADVANTAGES OF STEEL AS A CONSTRUCTION MATERIAL:-

- Steel is approximately ten times stronger that concrete. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for multistory buildings.
- Steel structures can be constructed very fast.
- Steel structures are ductile. Steel structures can be easily repaired and retrofitted to carry higher loads.
- Steel is also a very eco-friendly material.
- Since steel is produced in the factory under better quality control, steel structures have higher reliability and safety.

DISATVANTAGES OF STEEL:-

- Steel structures when placed in exposed conditions leads to corrosion. Hence they require frequent painting. They are not fire resistant.



Stress strain curve for mild steel **Stress strain curve for high strength steel**

Figure 16: Stress strain curves for mild steel and high strength steel

5.2 CONCRETE

Properties of Concrete: Plain concrete is prepared by amalgamation of cement, sand (fine aggregate), gravel (coarse aggregate) and water in precise proportions. Mineral admixtures may also be added to advance certain properties of concrete. Thus, the properties of concrete as regards to its strength and deformations depend on the individual properties of cement, sand, gravel, water and admixtures. Clauses 5 and 6 of IS 456:2000 specify the standards and requirements of the individual material and concrete, respectively. Plain concrete after preparation and placement needs curing to attain strength. However, plain concrete is very good in compression but weak in tension. That is why steel is used as reinforcing material to make the concrete sustainable in tension. Plain concrete, thus when reinforced with steel bars in appropriate locations is called as reinforced concrete.

The strength and deformation characteristics of concrete thus depend on the grade and type of cement, aggregates, admixtures, environmental conditions and curing. Pozzolanic materials (like fly ash cement) have slower rate of strength gain than ordinary Portland cement as recognized by code.

Characteristic strength property:

Characteristic strength is defined as the strength below which not more than 5% of the test results are expected to fall. Concrete is graded on the basis of its characteristic compressive strength of 150 mm size cube at 28 days and expressed in N/mm^2. The grades are designated by one letter M (for mix) and a number from 10 to 80 indicating the characteristic compressive strength (f_{ck}) in N/mm^2. As per IS 456 (Table 2), concrete has three groups as (i) ordinary concrete (M 10 to M 20), (ii) standard concrete (M 25 to M 55) and (iii) high strength concrete (M 60 to M 80).

Creep of concrete: Creep is another time dependent deformation of concrete by which it continues to deform, usually under compressive stress. Thus, the long term deflection will be added to the short term deflection to get the total deflection of the structure. Accordingly, the long term modulus E_{ce} or the effective modulus of concrete will be needed to include the effect of creep due to permanent loads.

Shrinkage of concrete: Shrinkage is the time reliant deformation, generally compressive in nature. The constituent of concrete, size of the member and environmental situation are the factors on which the total shrinkage of concrete depends. However, the total shrinkage of concrete is most subjective by the total amount of water present in the concrete at the time of mixing for a given humidity and temperature. The cement content, however, influences the total shrinkage of concrete to a lesser extent.

CHAPTER -6 DESIGN

6.1 Slab

6.1.1 Floor Slab

SLAB DESIGN(FLOOR)
DESIGN OF CRITICAL CASE AT POSITION 4
$fy=415N/mm^2$ $fck=40N/mm^2$
Lx=3.9m Ly=4.25m b=1000mm
Lx/Ly=1.0875
ASSUME TOTAL DEPTH= 200mm
DEAD LOAD= 0.2×25= $5KN/m^2$
LIVE LOAD= $5KN/m^2$
FINISHING LOAD= $1KN/m^2$
TOTAL FACTORED LOAD (w)=1.5× (5+5+1)= 16.5 KN/m^2
$Mx(+) = άx(+)×w×(Lx)^2$ = 0.04×16.5×3.9×3.9= 10.038 KNm/m
$My(+) = άy(+)×w×(Lx)^2$ = 0.035×16.5×3.9×3.9=8.78 KNm/m
$Mx(-) = άx(+)×w×(Lx)^2$ = 0.053×16.53.9×3.9=13.3 KNm/m
$My(-) = άy(+)×w× (Lx)^2$= .047×16.5×3.9×3.9=11.79 KNm/m
Xm = 0.48×180 = 86.4 mm
d = $(Mx(-)/(.138×Fck×b))^{0.5}$ = 49 mm
Adopt effective depth =180mm
Total depth = 200mm
Area of steel Axt along short span = 0.36×fck×b×Xm/(0.87×fy)
Axt = $3445.9mm^2$

ADOPT 22mm BARS AT 100 C/C SPACING
AREA = 3801.2.2 mm^2>$3445.9mm^2$ (MID SPAN AND EDGES)

AREA OF TENSION STEEL Aty ALONG LONG SPAN=0.5Fck/Fy×[1-(1-4.6My(-)/(Fck×b×d×d))$^{.5}$]×b×d
Aty=$183.04mm^2$
MINIMUM TENSION STEEL= .0012×d×b=$240mm^2$
ADOPT Aty=240mm2

ADOPT 8mm BARS AT 200mm c/c SPACING
AREA=$251mm^2$>$250mm^2$ (MID-SPAN AND EDGES)

CORNER DETALING
A=.75×Axt=$2865.9mm^2$
ADOPT 20mm BARS AT 110mm C/C spacing
B=C=.3375×AXT=$1432.95mm^2$
ADOPT 12mm BARS AT 100mm c/c spacing
D=0mm2

CHECK FOR SHEAR FORCE AT SHORT EDGE
MAXIMUM SHEAR FORCE=w×Lx/.5=32.175KN/m
NOMINAN SHEAR STRESS= .1938N/mm2
% tensile steel=.12%
FOR M40 AT .12% STEEL
SHEAR STRENGTH IN SLAB=1.3×.3=.39N/mm2>.1938N/mm2

DEFLECTION CHECK AT SHORT SPAN

$Lx/d<20\times MF1$ Astpro=3801.2mm2

$Fs=0.58\times Fy\times Astreq/Astpro=$ 218.2N/mm2 Astreq=3445.9mm2

$pt=Axt\times100/(b\times d)=2.1\%$

MF1=1.1

$Lx/d=3900/180=21.66<20\times MF1=22$

6.1.2 Roof Slab

SLAB DESIGN (ROOF)

DESIGN OF CRITICAL CASE AT POSITION 4 $Fck = 40N/mm^2$

Lx=3.9m Ly=4.25m $Fy = 415N/mm^2$

Lx/Ly=1.0875

ASSUME TOTAL DEPTH = 200 mm

DEAD LOAD = $0.2\times25=5$ KN/m^2

LIVE LOAD = 1.5 KN/m^2

FINISHING LOAD = 1 KN/m^2

TOTAL FACTORED LOAD (w) = $1.5\times(5+1.5+1) = 11.25$ KN/m^2

$Mx(+) = ά x(+)\times w\times(Lx)^2 = 0.04\times16.5\times3.9\times3.9 = 6.8445$ KNm/m b=1000mm

$My(+) = ά y(+)\times w\times(Lx)^2 = 0.035\times16.5\times3.9\times3.9 = 5.9889$ KNm/m

$Mx(-) = ά x(+)\times w\times(Lx)^2 = 0.053\times16.5\times3.9\times3.9 = 9.0689$ KNm/m $Xm=.48\times180=86.4mm$

$My(-) = ά y(+)\times w\times(Lx)^2 = 0.047\times16.5\times3.9\times3.9 = 8.0422$ KNm/m

$d = (Mx(-)/(0.138\times Fck\times b))^{0.5} = 40.93$ mm

AREA OF STEEL Axt ALONG SHORT SPAN = $0.36\times fck\times b\times Xm/(0.87\times fy)$

Axt = 3445.9 mm^2

ADOPT 22mm BARS AT 100 c/c SPACING

AREA=3801.2.2mm^2>3445.9mm^2 (MID SPAN AND EDGES)

Aty=124.7mm2

MINIMUM TENSION STEEL= $.0012\times d\times b=240mm^2$

ADOPT Aty=240mm^2

ADOPT 8mm BARS AT 200mm c/c SPACING

AREA=251mm^2>250mm^2 (MID-SPAN AND EDGES)

CORNRR DETALING

$A=.75\times Axt=2865.9mm^2$

ADOPT 20mm BARS AT 110mm C/C spacing

$B=C=.3375\times AXT=1432.95mm^2$

ADOPT 12mm BARS AT 100mm c/c spacing

D=0mm2

CHECK FOR SHEAR FORCE AT SHORT EDGE

MAXIMUM SHEAR FORCE=$w\times Lx/.5=32.175$KN/m

NOMINAN SHEAR STRESS= .1938N/mm^2

% tensile steel=.12%

FOR M40 AT .12% STEEL

SHEAR STRENGTH IN SLAB=$1.3\times.3=.39N/mm^2>.1938N/mm^2$

DEFLECTION CHECK AT SHORT SPAN

$Lx/d<20\times MF1$
$Fs=0.58\times fy\times Astreq/Astpro=$ 218.2N/mm2 $Astpro=3801.2mm^2$
$pt=Axt\times100/(b\times d)=2.1\%$ $Astreq=3445.9mm^2$
MF1=1.1
$Lx/d=3900/180=21.66<20\times MF1=22$

6.2 RCC Beam

RCC beam
M = 129KNm fy = 415MPa fck = 40MPa
D = 400mm B = 400mm d = 360mm
Assume cover of 40 mm
$Xu(lim) = 700d/(.87\times fy+1100)$
Xu(lim) = 172.47mm
Xu = 114.95mm
As $Xu<Xu(lim)$
Under reinforced section.
Pt(lim) = 2.5%
Pt(min) = 0.3657%
$Ast = M/(0.87\times fy(d-0.42Xu))$
$Ast=1146.18mm^2$
Provide 16mm barsnin tension zone
No. of bars $Mt=1146.18/\pi\times8\times8$
Mt=5.6
Hence provide 6 bars at a spacing of 80mm.
Pt=Ast/Bd=1.2735%
Compression reinforcement design
MR=40.7KNm
Xu=40.23mm
$Ast=401.2mm^2$
Pt=.4457% Hence provide 2,16mm bars

1) Shear check
Vu=109KN $\delta v=Vu/B\times d=1.21MPa$
$\delta c=.7423MPa$
$Vus=(\delta v-\delta c)\times B\times d$
Vus=42.08KN
Provide striups at 90°
Spacing Sv=170mm
Hence provide 8mm bars @ 170mm spacing

2)Development length
$Ld=.87\times Fy\times ø/4\delta bd$
Ld=473.06mm $\delta bd=1.9\times1.6=3.04MPa$
Provide 2 bent up bars at 45° inclination to horizontal.

3)Deflection check
$L/d=10.83<26$ OK

6.3 RCC Column

RCC Column
Degree of end restrain of the member.
Effectively held in position at both ends ,but not against rotation.
Effective length=L

P=4177KN	Mux=52.68KNm		Muy=1.34KNm
Fck=40MPa	Fy=415MPa		

Assume cover of 40mm
Let the dymensions of column be 600X600mm

D=600mm	B=600mm	d'=40mm	H=3.05m

Let assume %steel as 0.9%.

p/Fck=0.02	d'/D=0.066

1) Uniaxial moment capacity of section
P/Fck×b×D=1000×4177/(40×560×600)=0.31
Using chart 44 of BIS handbook for d'/D=0.066
$M/Fck\times b\times D^2=0.06$
$Mux1=0.06\times fck\times b\times D^2$
Mux1=518.4KNm
Muy1=518.4KNm
Pz=0.45Fck×Ac + 0.75×Fy×Asc
$Pz=0.45\times 40\times 600^2 + 0.75\times 0.009\times 415\times 600^2$
Pz=7488.4KN

P/Pz=0.557	Mux/Mux1=0.1	Muy/Muy1=0.002

ά=1+(0.357/.6)=1.595
$(Mux/Mux1)^{1.595} + (Muy/Muy1)^{1.595}=0.56<1$
Area of steel=0.009×600×600=3240mm²
Hence provide 28 , 12 mm bars

6.4 Steel Beam

STEEL BEAM
LET US USE ISMB 600

D(mm)=	600	NOW		
d(mm)=	519.4	b/T=	5.1724	<9.4£
	PLASTIC			
B(mm)=	210	AND		
R1(mm)=	20	d/t=	43.283	<84£
	PLASTIC			
R2(mm)=	10			
I_x(cm4)=	91800	£=(250/fy)^.5=		1
I_y(cm4)=	2650			
Z_e(cm3)=	3060.4	ßb=	1	
Z_p(cm3)=	3510.63			
t(mm)=	12	Vu(KN)=	252.59	

T(mm)= 20.3

f_y(Mpa)= 250 Mu(KNm)= 359.62

A(mm2)= 15600

b(mm)= 105 γmo= 1.1

NOW

d/t= 43.283333 <67£ SHEAR YIELDING

V_d(KN)= $Fy \times A/((\gamma mo) \times (3)^{0.5}))$= 2046.969 >Vu SAFE IN SHEAR

Vu< 0.6×Vd= 82<617.22

M_d(KNm)= $ß_b \times Z_p \times f_y/\gamma_{mo}$= 797.8705 >Mu OK

DEFLECTION CHECK

$\delta_{limiting}$(mm)= 12.5 L(mm)= 3000

δ_{max}(mm)=

LOCAL FAILURE CHECK

1) WEB BUCKLING RESISTANCE

LET US USE AN ANGLE SECTION OF DIMENSION 100×100×10(mm)

b1(mm)= 100 KL/r=2.5×d/t= 108.2083

n1(mm)= 200

fcd(MPa)= 114

P(KN)= (b1+n1)×t×fcd= 410.4 KN > 82KN OK

2)WEB CRIPPLING RESISTANCE

b1(mm)= 100

n2(mm)=2.5× (T+R1)= 100.75

$A \times f_y/\gamma_{mo}$= $(b1+n2) \times t \times f_y/\gamma_{mo}$= 547.5 KN > 82KN OK

6.5 Steel Column

Design Of Member No. 286 As Per IS:800

STEEL COLUMN

Input Parameters

Member Section SEC2

Cross Sectional Area A_x (m^2)	0.03
Shear Area Along Major Axis A_z (m^2)	0.03
Shear Area Along Minor Axis A_y (m^2)	0.03
r_z (m)	0.19
r_y (m)	0.09
Section Modulus About Major Axis - Tension Edge S_{tz} (m^3)	0.00
Section Modulus About Major Axis - Compression Edge S_{zz} (m^3)	0.00
Section Modulus About Minor Axis - Tension Edge S_{ty} (m^3)	0.00
Section Modulus About Minor Axis - Compression Edge S_{yy} (m^3)	0.00
Unsupported Length - Major Axis For Slenderness Check L_z (m)	3.05
Unsupported Length - Minor Axis For Slenderness Check L_y (m)	3.05
Effective Length For Allowable Bending Stress Calculation Unl (m)	3.05
Yield Stress f_y (MPa)	250.00
Allowable Ratio For Interaction Check	1.00

Design Forces

Combined Axial Force & Bi-axial moment

Axial Load F_x (kN)	1587.53
Major Axis Moment M_z (kNm)	2.94
Minor Axis Moment M_y (kNm)	-99.60
Critical Loadcase No.	23
Critical Section (m)	3.05
Beam No.	2325

Shear Along Major Axis

Shear Along Major Axis F_z (kN)		52.94
Critical Loadcase No.		23
Critical Section (m)		0.00
Beam No.		2518
Shear Along Minor Axis		
Shear Along Minor Axis F_y (kN)		57.83
Critical Loadcase No.		22
Critical Section (m)		0.00
Beam No.		2518

Details Of Calculation

Slenderness Checking

L_{ez} (m)		3.05
L_{ey} (m)		3.05
L_{ez} / r_z		16.41
L_{ey} / r_y		34.39
Actual Slenderness Ratio		34.39
Allowable Slenderness Ratio		180.00
Status		SAFE

Check Against Axial Compression And Bi-Axial Bending

Actual Compressive Stress f_c (MPa)	$=F_x / A_x$	63.28

Calculation Of Allowable Compressive Stress :

E (MPa)		200000.00
L / r		34.39
f_{cc} (MPa)	$\square\square\square^{2}$ E / $\square\square^{2}$	1669.34
Allowable Compressive Stress $f_{c\_allowable}$ (MPa)	$=0.6 f_{cc} f_y / [(f_{cc})^{1.4} + (f_y)^{1.4}]$	142.91

Bending about Major Axis :

Actual Bending Compressive Stress- Major Axis f_{cz} (MPa)	$= Mz / S_{zz}$	0.76

Calculation of Allowable Bending Stress About Major Axis :

L / r		34.39
Y (MPa)	$=26.5 \times 10^5 / (L / r)^2$	2241.09
D / T_f		1.#J
X (MPa)	$=Y[1 + 1/20((L / r) / (D / T_f))^2]^{0.5}$	2241.09
k_1		1.00
k_2		0.00
c_1		1.00
c_2		1.00
Elastic Critical Stress f_{cb} (MPa)	$=k_1(X + k_2 Y) c_2 / c_1$	2241.09
Allowable Bending Compressive Stress- Major Axis $f_{cz\_allowable}$ (MPa)	$=0.66 f_{cb} f_y / [(f_{cb})^{1.4} + (f_y)^{1.4}]^{1/1.4}$	159.71

Bending about Minor Axis :

Actual Bending Compressive Stress- Minor Axis f_{cy} (MPa)	$=My / S_{yy}$	63.09

Calculation of Allowable Bending Stress About Minor Axis :

Allowable Bending Compressive Stress - Minor Axis $f_{cy\_allowable}$ (MPa)	$=0.66f_y$	165.00

Interaction Checking

Interaction ratio	$=f_c / 0.60f_y + f_{cz} / f_{cz\_allowable} + f_{cy} / f_{cy\_allowable}$	0.81
Status		SAFE

Check Against Shear

Shear Stress Along Major Axis V_z (MPa)	$=F_z / A_z$	2.11
Shear Stress Along Minor Axis V_y (MPa)	$=F_y / A_y$	2.31
Allowable Shear Stress $V_{\_allowable}$ (MPa)	$=0.4f_y$	100.00
Interaction ratio (Along Major Axis)	$=V_z / V_{\_allowable}$	0.02
Status		SAFE
Interaction ratio (Along Minor Axis)	$=V_y / V_{\_allowable}$	0.02
Status		SAFE

6.6 Connection Design

6.6.1 Beam to column connection

a)Beam To Column Connections

M = 290 KNm

V = 220 KN

Provide seat and cleat angle of length equal to width of bem flange which is 210mm.

fu = 410 Mpa

fy = 250 MPa

ϒm0 = 1.1

tw = 12 Mm

rb= 10 Mm

tf = 20.3 Mm

Let us provide clearance of 10mm.

g = 10 Mm

b = R×ϒm0/(tw×fyw) = 80.6667 Mm

b1 = b-(tf+rb) = 50.3667 Mm

Considering b provide angle section of 90×90×10mm.

R1 = 8.5 Mm

ta = 10 Mm

ra = 8.5 Mm

b2 = b1+g-(ta+ra) = 41.8667 Mm

Provide weld section and assume site welding.

ϒmw = 1.5

t = 10 Mm Min. of tf and ta

Max. size of weld,a = 3t/4 = 7.5 Mm

Min. size of weld,a = 3 Mm

Provide 7mm size of weld.

a = 7 Mm

te = a/$2^{0.50}$ = 4.94975 Mm

End turns > 2a 14 Mm On each side

Rw = te×fu/($3^{0.50}$×ϒmw) = 781.115 KN/m

Lw = R/(2×Rw) = 140.824 Mm

Provide total weld length of 150mm.

Provide fillet weld with weld length of 180mm (90mm on each side in vertical direction) and 15mm of end turns on each side.

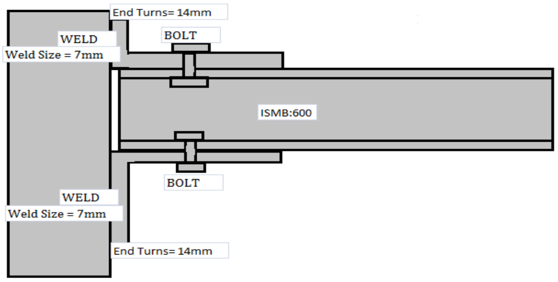


Figure 17 :beam to column connection

b)Design Of Bolts (M24 size and 8.8 grade)
P = 160 KN
Let us use M20, 8.8 grade bolts.
d = 24 mm
fub = 800 MPa
fyb = 640 MPa
fup = 400 MPa
Let us provide clearance of 2mm.
dh = 26 mm
Min. edge distance,emin. = 1.50×dh = 39 mm
Min. pitch, pmin. = 50 mm
ϒmb = 1.25
We are using HSFG. There are three types of forces acting.
1. Bearing force acting on bolts and on plates
Kb = 0.5 Calculated considering mi. pitch and min. edge distance
Asb = 452.571429 mm^2
Anb = 0.78×Asb = 353.006 mm^2
t = 10 mm
Vnpb = 2.50×Kb×d×t×fub = 120 KN
Vpb = Vnsf/ϒmw = 96 KN
2. Shear force acting on bolts due to friction.
μ = 0.5
ne = 1
Kh = 1
F0 = 0.70×fub×Anb = 197.683 KN
Vnsf = μ×ne×Kh×F0 = 98.8416 KN
Vsf = Vnsf/ϒmw = 79.0733 KN
3. Tension force acting on bolts.
Tnb1 = 0.90×fub×Anb = 254.164 KN
Tnb2 = fyb×Asb×ϒmb/ϒmo = 329.143 KN
Tnb = 254.1641 KN (Min. of two)
Tb = Tnb/ϒmb = 203.331 KN
Design force acting on bolts, V = 79.0733 K N (Min. of three forces)
No. of bolts,n = P/V = 2.02344 bolts

Use 2 no. of bolts 1 on each side of the web.
Checks Performed:
1. Combined shear and tension. Passed
2. Block failure check. Passed
Conclusion:
1. Provide edge distance of 40mm.
2.No need to provide pitch. Provide gauge distance of 80mm.
3. The total lap between seat or cleat angle with beam (ISMB-600) is 80mm.
4. Total length of seat and cleat angle is 90mm. Suitable

6.6.2 Column to column connection

Column To Column Connections
P =100KN
Provide seat and cleat angle of length equal to width of bem flange which is 210mm.
fu =400Mpa
fy =250MPa
ϒm0 =1.1
Provide a plate splice between two columns of width 250mm and thickness 12mm. Length can be determined from the length of fillet weld.
Provide fillet weld and assume site welding.
ϒmw =1.5
t =12mm Depending on thickness of plate joined.
Max. size of weld,a = t-1.50 =10.5mm
Min. size of weld,a =3mm
Provide 10mm size of weld.
a =10mm
te = a/2^0.50 =7.07mm
Pdw = Lw×te×fu/(3^0.50×ϒmw)
Rw = te×fu/(3^0.50×ϒmw) =1088.5KN
Lw2 = d =250mm
Pdw2 =272.124KN
As such no need to provide extra length but provide a splice.
Simply join the two columns. Also provide extra 200×250×12mm splice.

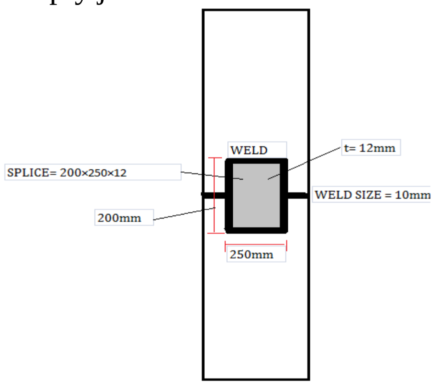


Figure 18: column to column connection

6.6.3 Base Plate

BASE PLATE

P=70KN
M=85KNm
fck=40N/mm2
Grade of steel 410

1) Size of base plate
e=P/M=85×1000/70=1214mm
Let us assume L=720mm
720mm<6e
B=2×P/L×0.45×fck=10.8mm
Let us take B=400mm
Provide a rectangular plate of 720x400
Area=720×400=288000mm2
Z=400×720×720/6=34560000mm3
Maximum Pressure:
p=(70000/288000)+(85/34.65)=2.7N/mm2

2)Thickness of plate
Let us assume the compound section is placedat 140mm from one side of plate
4 boltes are used at 50mm from the corners

Base pressure at section =(720-140)×2.7/720=2.17N/mm2
Moment at section=(2.17×140×140/2)+(2.7-2.17)×140×140/3=24700Nmm
Moment capacity=1.2×fy×Ze/γ
Ze=t²/6
t=√24700/45.45=24mm

3) Weld connection beam-column to base plate
Compound section ,Area=225.86cm2
Z=1474.66cm3
Axial stress=70000/22586=3.1N/mm2
Bending stress=85000/1417.66=57.6N/mm2
On the basis of elastic stress distribution, there is a compressive stress over the entire base.
The base plate and column are to be machined together so a weld is requiredto hold the plate in position.
Use a 6mm continuous fillet weldaround the column profile.

6.7 Foundation Design

1)Pile Geometric data
Pile Cap Length P_{CL} = 4.000 m

Pile Cap Width P_{CW} = 2.500 m

Initial Pile Cap Thickness t_I = 0.300 m

2)Pile capacity

Axial Capacity P_P = 500.000 kN

Lateral Capacity P_L = 100.000 kN

Uplift Capacity P_U = 300.000 kN

3) Material property

Concrete f'_c = 25000.004 kN/m^2

Reinforcement f_y = 415000.070 kN/m^2

Table 4:Pile reaction

–	Arrangement		Reaction		
Pile No.	X (m)	Y (m)	Axial (kN)	Lateral (kN)	Uplift (kN)
1	-1.500	-0.750	-670.773	3.825	0.000
2	-1.500	0.750	-659.820	3.825	0.000
3	0.000	-0.750	-667.444	3.825	0.000
4	0.000	0.750	-656.491	3.825	0.000
5	1.500	-0.750	-664.115	3.825	0.000
6	1.500	0.750	-653.162	3.825	0.000

5) Reinforcement calculation
Maximum bar size allowed along length # 40

Maximum bar size allowed along width # 20

Bending Moment At Critical Section = -1596.683 kNm (Along Length)

Bending Moment At Critical Section = -901.033 kNm (Along Width)

Pile Cap Thickness t = 1.280 m

Selected bar size along length # 12

Selected bar size along width # 12

Selected bar spacing along length = 68.23 mm

Selected bar spacing along width = 73.36 mm

6)Check for moment

Effective Depth(d_{ef}) = 1.149m

Depth of neutral axis for balanced section(x_u)= 0.550m

As Per IS 456 2000 ANNEX G,G-1.1 C

Ultimate moment of resistance(M_{ulim}) = = 11394.960 kNm

We observed M_u <= M_{ulim} hence singly reinforced and under reinforced section can be used

7) Check for shear

Design Shear Force for One-Way Action V_u = -486.630 kN

As Per IS 456 2000 ANNEX B,B-5.1 and Clause No

34.2.4.2

Design Shear Stress (T_v) = = -169.410 kN/m^2

Allowable Shear Stress (T_c) = = 276.237 kN/m^2

Where Beta = = 21.714

and percentage of steel required (p_t) = = 0.134

Here $T_v <= T_c$ Hence safe

8) Check for two way shear

Design Two-Way Shear force = -1810.616 kN

As Per IS 456 2000 Clause 31.6.2.1

Two Way Shear Stress(T_v) = = -225.246 kN/m^2

Where,perimeter of critical section(b_0) = = 6.996 m

As Per IS 456 2000 Clause 31.6.3.1

Allowable shear stress = = 1250.000 kN/m^2

Where,k_s = = 1.000

Ratio of shorter to longer dimension(B_c) = 1.000

and,T_c = 1250.000 kN/m^2

$0.25 \times \sqrt{f_c} \times b \times d$

$T_v < K_sT_c$ hence Safe.

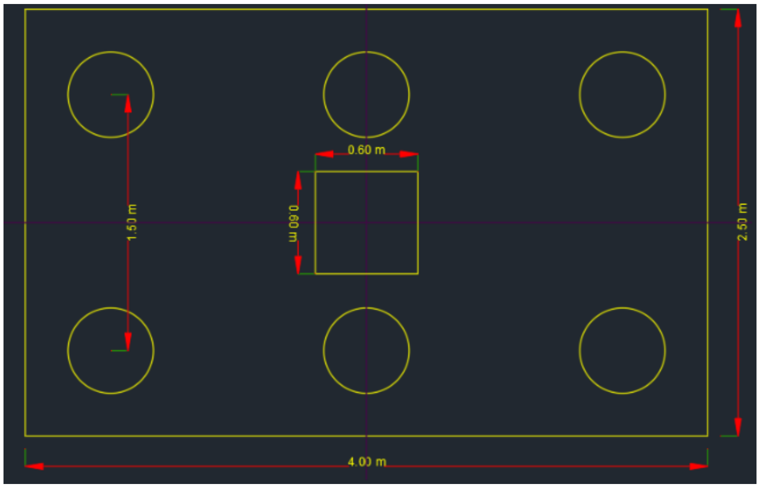


Figure 19: Foundation Top view

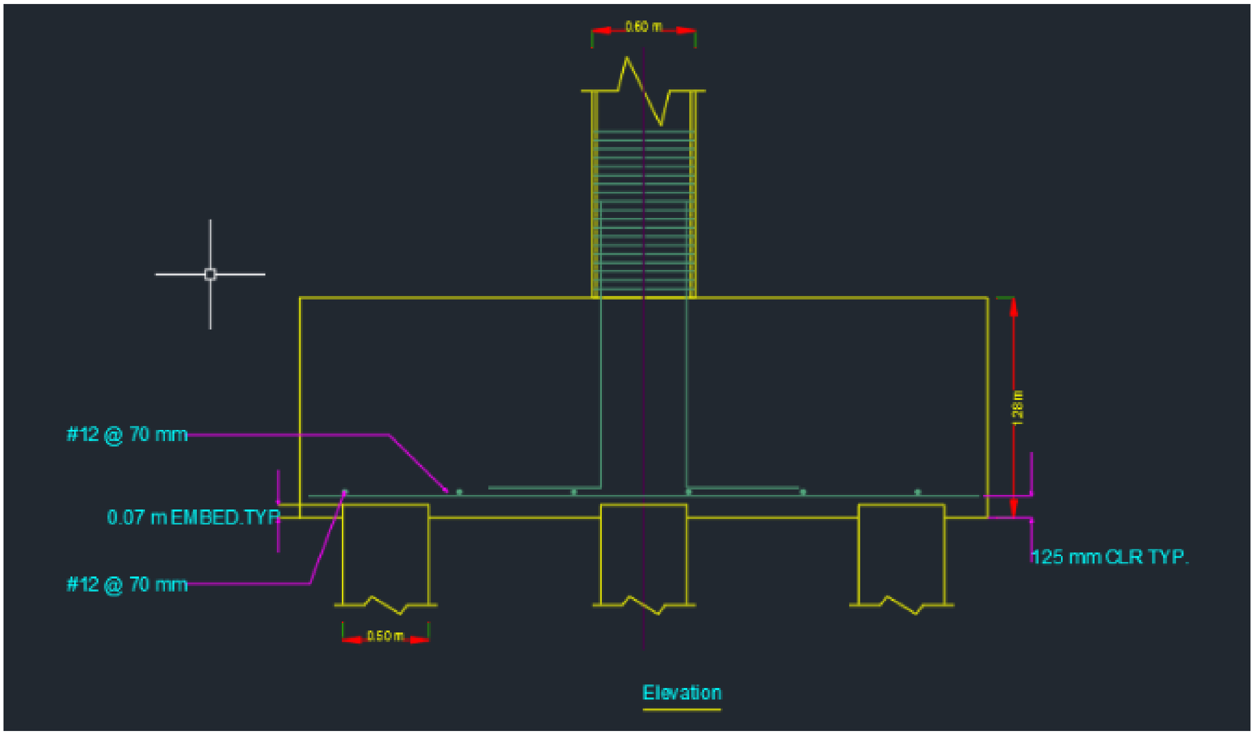


Figure 20: Foundation Elevation

CHAPTER 7 MASONRY INFILLS

7.1 Introduction

Significant experimental and analytical research effort has been expended till date in understanding the behavior of masonry infilled frames [CEB, 1996]. Infills interfere with the lateral deformations of the RC frame; separation of frame and infill takes place along one diagonal and a compression strut forms along the other. Thus, infills add lateral stiffness to the building. The structural load transfer mechanism is changed from frame action to predominant truss action (Figure 1); the frame columns now experience increased axial forces but with reduced bending moments and shear forces.

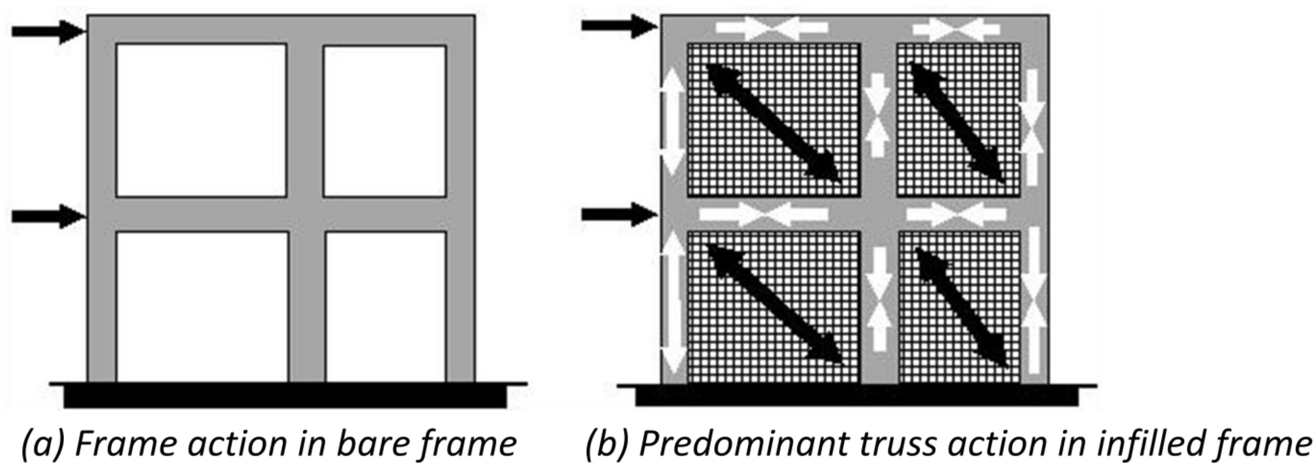


Figure 21: Masonry infills effect

When infills are non-uniformly placed in plan or in elevation of the building, a hybrid structural load transfer mechanism with both frame action and truss action, may develop. In such structures, there is a large concentration of ductility demand in a few members of the structure. For instance, the *soft-storey effect* (when a storey has no or relatively lesser infills than the adjacent storeys), the *short-column effect* (when infills are raised only up to a partial height of the columns), and *plan-torsion effect* (when infills are unsymmetrically located in plan), cause excessive ductility demands on frame columns and significantly alter the collapse mechanism. Another serious concern with such buildings is the out-of-plane collapse of the infills which can be life threatening. Even when the infills are structurally separated from the RC frame, the separation may not be adequate to prevent the frame from coming in contact with the infills after some lateral displacement; the compression struts may be formed and the stiffness of the building may increase.

Infills possess large lateral stiffness and hence draw a significant share of the lateral force. When infills are strong, strength contributed by the infills may be comparable to the strength of the bare frame itself. The mode of failure of an infilled building depends on the relative strengths of frame and infill. And, its ductility depends on the (a) infill properties, (b) relative strengths of frame and infill, (c) ductile detailing of the frame when plastic hinging in the frame controls the failure, (d) reinforcement in the infill when cracking in infills controls the failure, and (e) distribution of infills in plan and elevation of the building.

Masonry infills in reinforced concrete buildings cause several undesirable effects under seismic loading: short-column effect, soft-storey effect, torsion, and out-of-plane collapse. Hence, seismic codes tend to discourage such constructions in high seismic regions. However, in several moderate earthquakes, such buildings have shown excellent performance even though many such buildings were not designed and detailed for earthquake forces.

7.1.1 ADVANTAGES:

A) Stiffness:

Average initial stiffness of infilled RC frame is about *4.3* times that of bare frame when masonry is unreinforced, and about *4.0* times that of bare frame when masonry is reinforced. This difference is explained by the inadvertent increase in mortar thickness in every third brick course in reinforced masonry where the reinforcement is provided. URM infill with inclined brick courses (MRF23) caused the smallest increase in stiffness relative to the frames with horizontal brick courses; the unduly large thickness of mortar and smaller cut-pieces of bricks used in this specimen explain the smaller increase.

B) Strength:

On an average, URM infilled frames have about *70%* higher strength than the bare frames; the value is about *50%* higher in case of RM infilled frames. The increase is the largest when the brick courses are inclined to the horizontal, since these inclined brick courses prevent the formation of weak horizontal planes of sliding observed with horizontal brick courses.

C) Ductility:

Under cyclic loading, the bare frame failed in the first *25mm* displacement cycle; this may have been an exceptional case. On the other hand, the bare frame under monotonic loading sustained up to *47.8mm* displacement while the infilled frames under cyclic loading sustained three cycles of *40mm* displacement excursion and failed in the first excursion to *50mm*. Thus, the deformability of bare and infilled frames is quite comparable. The yield displacement of infilled frames is much smaller than that of the bare frame, and hence, the infilled frames have a considerable larger ductility. Further, as expected, addition of reinforcement in infills increases the ductility of infilled frames. The average ductility of URM infilled frames is about *4.0* times that of the bare frames; ductility of RM infilled frame is about *5.1* times that of the bare frames.

7.2 Infill model

Infill Model

Modulus of elasticity and Youngs modulus of elasticity of concrete,Ec= 2.1718×10^10 Pa

Compressive strength of brick,fm= 10MPa

Column dimensions= 600mm×600mm 500mm×500mm 400mm×400mm

Moment of inertia of column,I= 0.0108m^4 0.005208m^4 0.002133m^4

Modulus of elasticity and Youngs modulus of elasticity of infill,Em=500×fm= 5500MPa

Table 5: Infill model

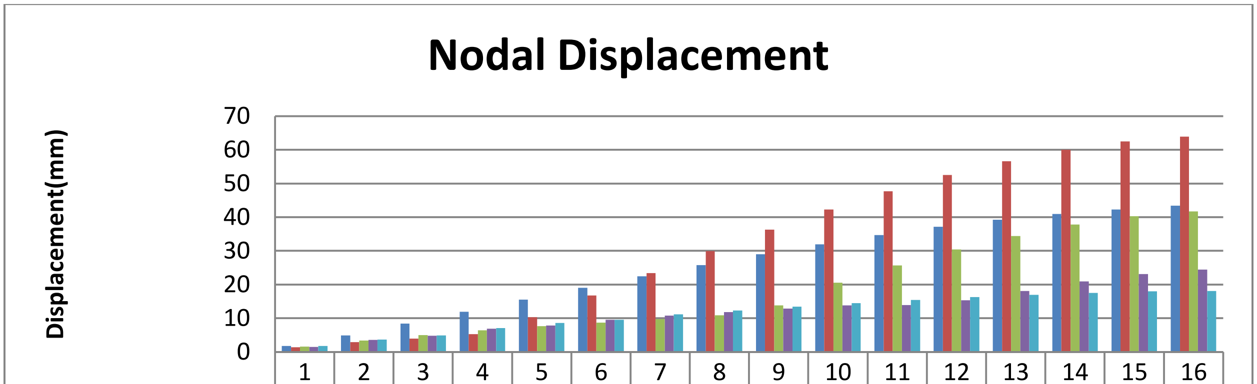
S.No.	hcolumn(m)	Beam(m)	hinf=hcol-tb(m)	L(m)	tcolumn(m)	Linf=L-tcol(m)	I(m^4)	Em(MPa)
Column Dimension 600mm×600mm								
1	3.05	0.4	2.65	3.9	0.6	3.3	0.0108	5500
2	3.05	0.4	2.65	3	0.6	2.4	0.0108	5500
3	3.05	0.4	2.65	4.25	0.6	3.65	0.0108	5500
Column Dimensions 500mm×500mm								
1	3.05	0.4	2.65	3.9	0.5	3.4	0.005208	5500
2	3.05	0.4	2.65	3	0.5	2.5	0.005208	5500
3	3.05	0.4	2.65	4.25	0.5	3.75	0.005208	5500
Column Dimensions 400mm×400mm								
	3.05	0.4	2.65	3.9	0.4	3.5	0.002133	5500
	3.05	0.4	2.65	3	0.4	2.6	0.002133	5500
	3.05	0.4	2.65	4.25	0.4	3.85	0.002133	5500

Ec(MPa)	rinf(m)	tinfill(m)	TanΘ	Sin(2Θ)	λ(m^-1)	bstrut(m)	Astrut=bstrut×tinfill(m^2)
Column Dimension 600mm×600mm							0
21718	4.95101	0.2	0.80303	0.976413	0.8107169	**0.603204027**	0.120640805
21718	4.278142	0.2	1.104167	0.995111	0.8145704	**0.520237754**	0.104047551
21718	5.231157	0.2	0.726027	0.950848	0.8053572	**0.639028771**	0.127805754
Column Dimensions 500mm×500mm							
21718	4.95101	0.2	0.779412	0.96973	0.9712067	**0.561160702**	0.11223214
21718	4.278142	0.2	1.06	0.998305	0.9782836	**0.483489898**	0.09669798
21718	5.231157	0.2	0.706667	0.942613	0.9643449	**0.594597234**	0.118919447
Column Dimensions 400mm×400mm							
21718	4.95101	0.2	0.757143	0.962511	1.2117708	**0.513622117**	0.102724423
21718	4.278142	0.2	1.019231	0.999819	1.2233461	**0.442133666**	0.088426733
21718	5.231157	0.2	0.688312	0.934081	1.2027218	**0.544314333**	0.108862867

7.3 Studying the effects of infills

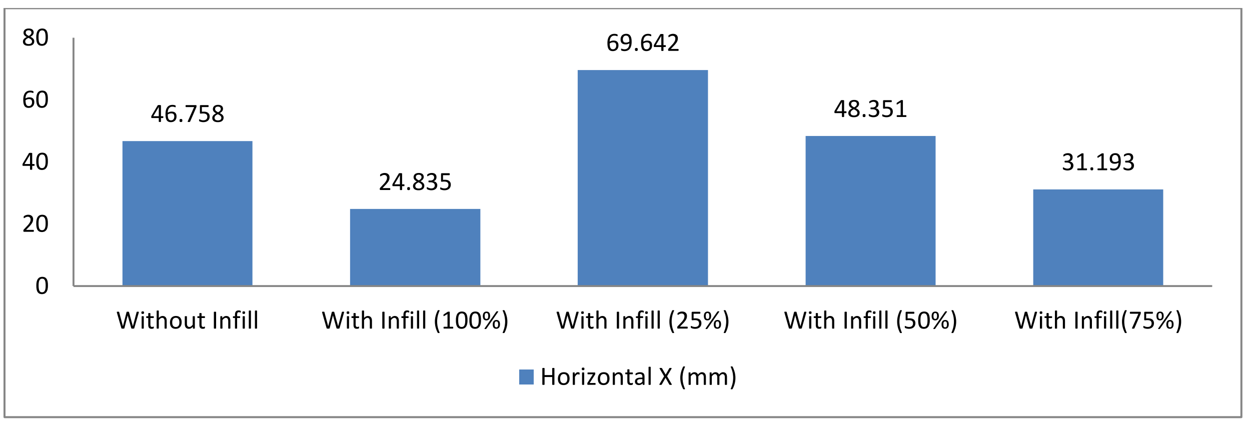
The effect of infills on a structure is studied providing struts .thus five different structures were created i.e. without infills , 25% infills , 50 % infills .75% infills , 100% infills , and the effects of the infills were studied on displacement , bending moment , shear force ,and story drift.

7.4 Nodal displacement

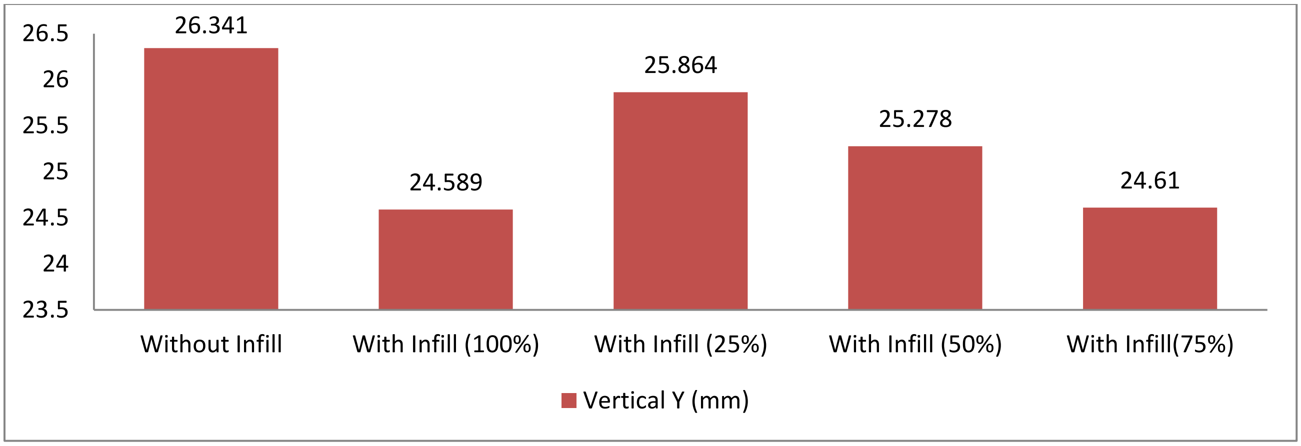


	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Without Infill	1.754	4.9	8.39	11.96	15.52	19.04	22.48	25.81	28.98	31.96	34.71	37.15	39.27	40.99	42.27	43.45
With Infill (25%)	1.42	2.94	3.98	5.24	10.32	16.75	23.41	29.99	36.31	42.24	47.67	52.5	56.63	60	62.5	63.93
With Infill (50%)	1.6	3.356	4.963	6.415	7.69	8.661	9.71	10.9	13.84	20.57	25.7	30.38	34.45	37.78	40.27	41.72
With Infill (75%)	1.5	3.54	4.8	6.9	7.86	9.6	10.77	11.81	12.85	13.8	13.95	15.32	18.05	20.91	23.1	24.41
With Infill (100%)	1.729	3.642	4.95	7.065	8.576	9.598	11.21	12.35	13.4	14.47	15.43	16.27	16.97	17.54	17.95	18.12

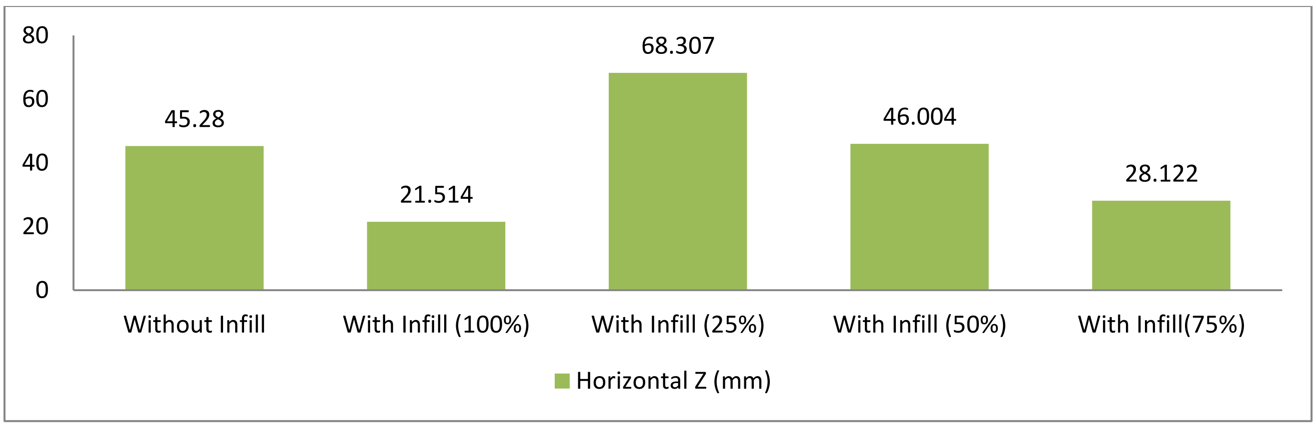
Graph 1: Nodal displacement



Graph 2 : Displacement in X direction



Graph 3 : Displacement in Y direction



Graph 4 : displacement in Z direction

Table 6 : *% Difference In Nodal Displacement From Previous Value In z- Direction*

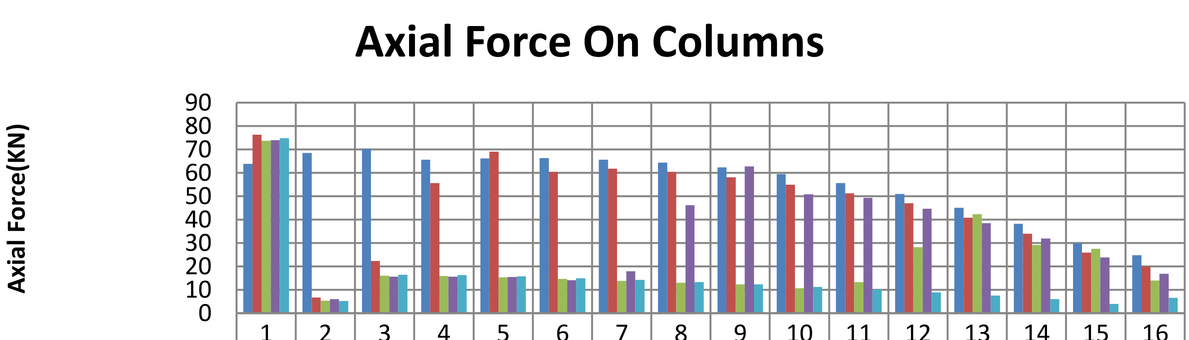
10	Without Infill	With Infill (25%)	With Infill (50%)	With Infill (75%)	With Infill (100%)
1	179.36%	107.04%	109.75%	136.00%	110.64%
2	71.22%	35.37%	47.88%	35.59%	35.91%
3	42.55%	31.66%	29.26%	43.75%	42.73%
4	29.77%	97.00%	19.88%	13.91%	21.39%
5	22.69%	62.26%	12.63%	22.14%	11.92%
6	18.06%	39.77%	12.11%	12.19%	16.84%
7	14.81%	28.11%	12.26%	9.66%	10.11%
8	12.28%	21.06%	26.97%	8.81%	8.54%
9	10.28%	16.33%	48.59%	7.39%	7.99%
10	8.60%	12.86%	24.97%	1.09%	6.62%
11	7.03%	10.13%	18.21%	9.82%	5.42%
12	5.71%	7.87%	13.40%	17.79%	4.35%
13	4.38%	5.95%	9.67%	15.89%	3.33%

14	3.12%	4.17%	6.58%	10.45%	2.35%
15	2.80%	2.29%	3.60%	5.68%	0.90%

Table 7: Change In Nodal Displacement In z- Direction

Node No.	Storey	With Infill (25%)	With Infill (50%)	With Infill (75%)	With Infill (100%)
8	1	19.04%	14.48%	14.48%	1.43%
177	2	40.00%	27.76%	27.76%	25.67%
247	3	52.56%	42.79%	42.79%	41.00%
317	4	56.19%	42.31%	42.31%	40.93%
387	5	33.49%	49.36%	49.36%	44.74%
457	6	12.03%	49.58%	49.58%	49.59%
527	7	-4.15%	52.09%	52.09%	50.12%
597	8	-16.21%	54.24%	54.24%	52.16%
687	9	-25.29%	55.66%	55.66%	53.75%
737	10	-32.17%	56.82%	56.82%	54.72%
807	11	-37.34%	59.81%	59.81%	55.54%
877	12	-41.32%	58.76%	58.76%	56.21%
947	13	-44.21%	54.05%	54.05%	56.78%
1017	14	-46.38%	48.98%	48.98%	57.21%
1087	15	-47.86%	45.36%	45.36%	57.53%
1157	16	-47.12%	43.83%	43.83%	58.31%

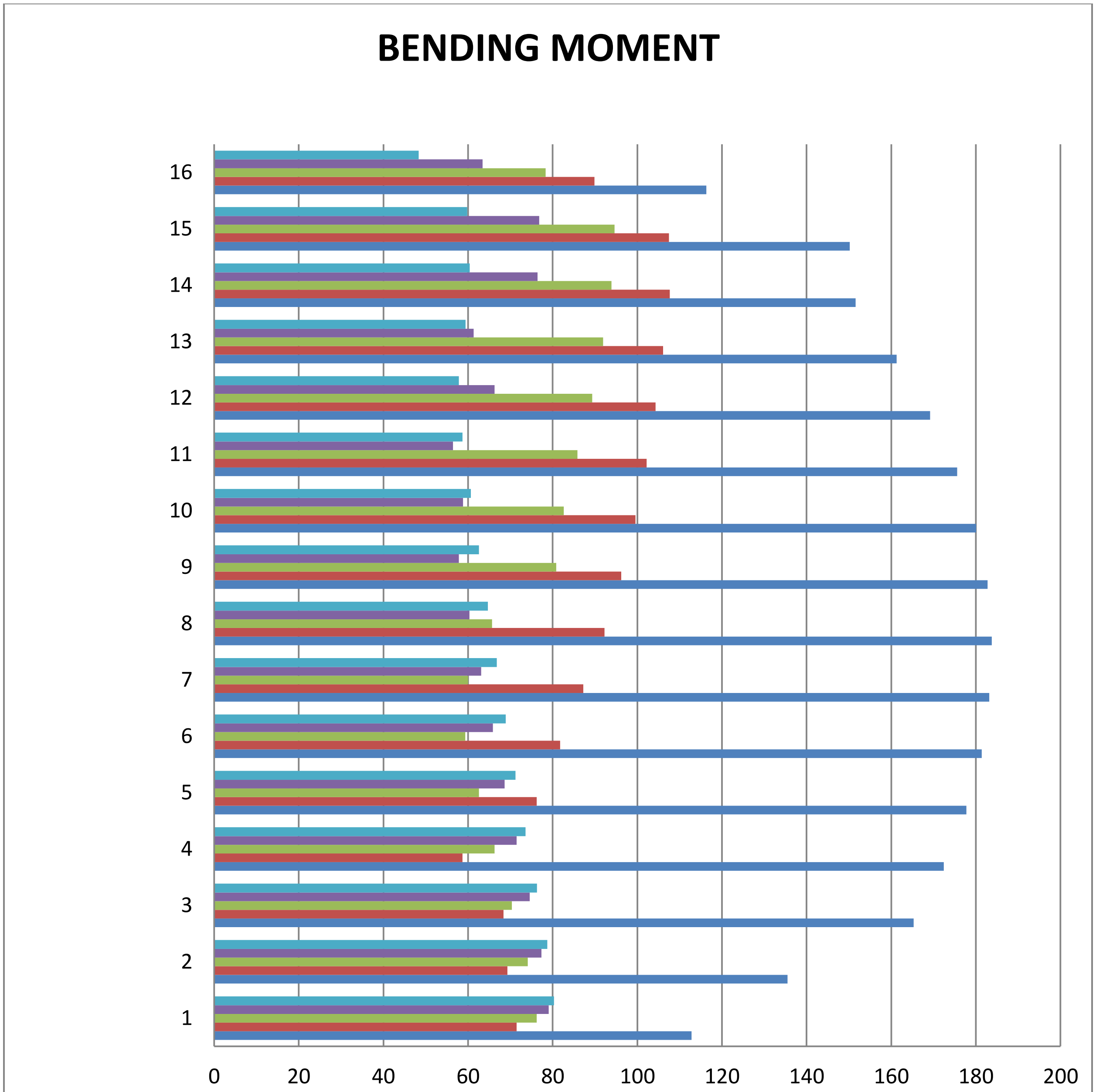
7.5 Axial force



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Without Infill	63.92	68.58	70.35	65.63	66.26	66.31	65.73	64.44	62.38	59.49	55.7	50.94	45.14	38.26	29.85	24.83
With Infill (25%)	76.29	6.7	22.33	55.65	69.08	60.46	61.84	60.44	58.13	54.94	51.3	46.97	40.84	34.03	25.9	20
With Infill (50%)	73.75	5.42	16.02	15.95	15.38	14.65	13.92	13.07	12.29	10.73	13.32	28.24	42.31	29.14	27.53	14
With Infill (75%)	74.04	6.064	15.6	15.68	15.5	14.15	17.99	46.15	62.81	50.89	49.41	44.7	38.58	31.93	23.86	16.81
With Infill (100%)	74.81	5.211	16.45	16.32	15.73	14.97	14.19	13.33	12.37	11.31	10.15	8.875	7.483	6.032	3.979	6.563

Graph 5 : Axial force

7.6 Bending moment



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
With Infill (100%)	80.31	78.76	76.3	73.6	71.2	68.9	66.78	64.7	62.6	60.65	58.67	57.8	59.4	60.4	59.8	48.3
With Infill (75%)	79.03	77.36	74.6	71.5	68.64	65.85	63.1	60.3	57.8	58.8	56.46	66.29	61.35	76.4	76.8	63.44
With Infill (50%)	76.25	74.1	70.34	66.24	62.57	59.34	59.99	65.7	80.84	82.59	85.87	89.34	91.9	93.9	94.6	78.3
With Infill (25%)	71.5	69.3	68.4	58.7	76.2	81.75	87.25	92.25	96.23	99.55	102.2	104.3	106.1	107.7	107.5	89.84
Without Infill	112.8	135.5	165.3	172.4	177.8	181.4	183.2	183.8	182.8	180.1	175.6	169.2	161.3	151.6	150.2	116.3

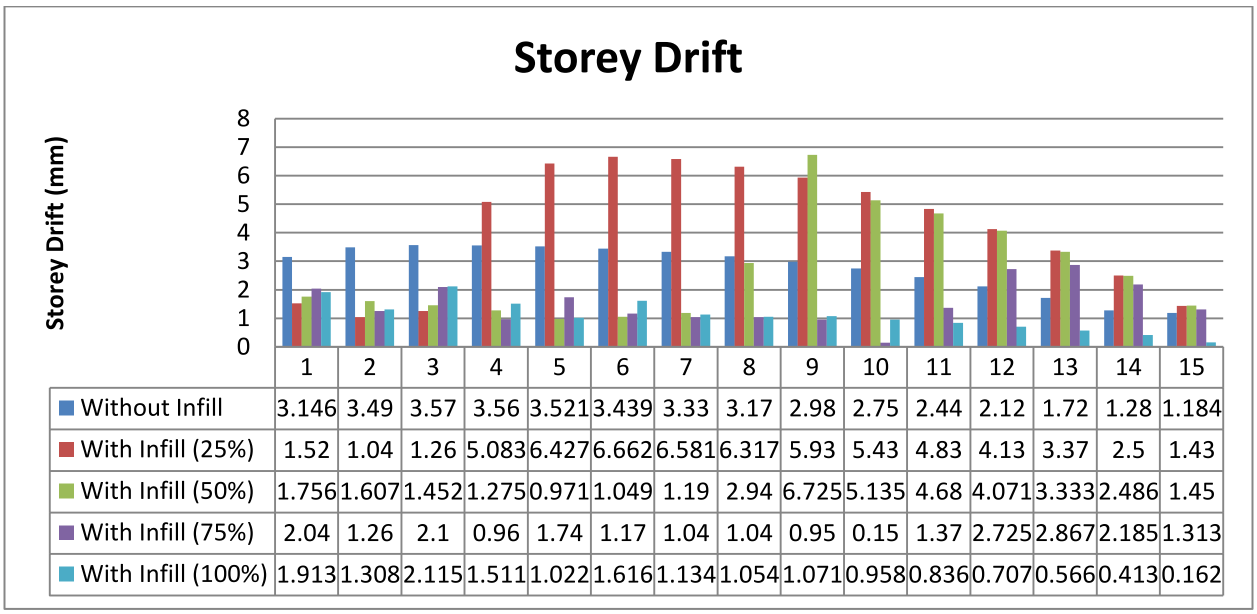
Mz(KNm)

Graph 6 : Bending moment

Table 8 : % Change In Bending Moment for Beams Mz(KNm)

Beam No.	Storey	% change for 25%	% change for 50%	% change for 75%	% change for 100%
118	1	36.61%	32.40%	29.94%	28.80%
332	2	48.85%	45.31%	42.90%	41.87%
525	3	58.63%	57.45%	54.88%	53.85%
718	4	65.95%	61.58%	58.53%	57.31%
911	5	57.14%	64.81%	61.39%	59.96%
1104	6	54.93%	67.29%	63.70%	62.02%
1297	7	52.37%	67.25%	65.56%	63.55%
1490	8	49.81%	64.25%	67.19%	64.80%
1683	9	47.36%	55.78%	68.38%	65.75%
1876	10	44.73%	54.14%	67.35%	66.32%
2069	11	41.80%	51.10%	67.85%	66.59%
2262	12	38.36%	47.20%	60.82%	65.84%
2455	13	34.22%	43.03%	61.97%	63.17%
2648	14	28.98%	38.06%	49.60%	60.16%
2841	15	28.45%	37.02%	48.87%	60.19%
3034	16	22.75%	32.67%	45.45%	58.47%

7.7 Story drift

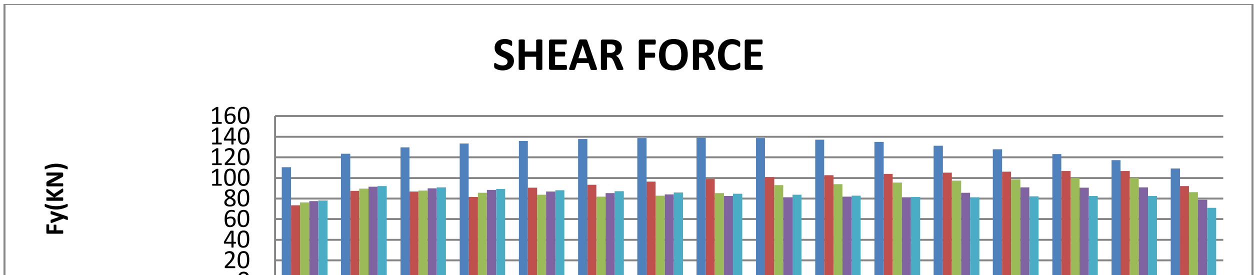


Graph 7 : Story drift

Table 9 : % Change In Story drift

Floor	With Infill (25%)	With Infill (50%)	With Infill (75%)	With Infill (100%)
1	51.68%	44.18%	35.16%	39.19%
2	70.20%	53.95%	63.90%	62.52%
3	64.71%	59.33%	41.18%	40.76%
4	-42.78%	64.19%	73.03%	57.56%
5	-82.53%	72.42%	50.58%	70.97%
6	-93.72%	69.50%	65.98%	53.01%
7	-97.63%	64.26%	68.77%	65.95%
8	-99.27%	7.26%	67.19%	66.75%
9	-98.99%	-125.67%	68.12%	64.06%
10	-97.45%	-86.73%	94.55%	65.16%
11	-97.95%	-91.80%	43.85%	65.74%
12	-94.81%	-92.03%	-28.54%	66.65%
13	-95.93%	-93.78%	-66.69%	67.09%
14	-95.31%	-94.22%	-70.70%	67.73%
15	-20.78%	-22.47%	-10.90%	86.32%
Total	-830.57%	-171.60%	495.48%	939.46%
Avg.	-55.37%	-11.44%	33.03%	62.63%

7.8Shear force



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Without Infill	110	123	130	133	136	138	139	139	139	137	135	131	128	123	117	109
With Infill (25%)	73.5	87.3	86.9	81.5	90.7	93.5	96.3	98.9	101	103	104	105	106	107	107	92.2
With Infill (50%)	76.1	89.7	87.7	85.6	83.7	82	82.7	85.4	93.1	93.9	95.6	97.4	98.7	99.8	99.9	86.3
With Infill (75%)	77.6	91.3	89.9	88.3	86.9	85.4	83.9	82.6	81.2	81.8	81.2	85.7	90.9	90.7	91	78.7
With Infill (100%)	78.2	92.1	90.8	89.4	88.2	87	85.9	84.8	83.7	82.7	81.7	81.3	82.1	82.6	82.4	71.1

Graph 8 : Shear force

Table 10: Change In Shear force Fy (KN)

Beam No.	Storey	% change for 25%	% change for 50 %	% change for 75%	% change for 100%
118	1	33.46%	31.08%	29.77%	29.17%
332	2	29.28%	27.38%	26.03%	25.41%
525	3	32.95%	32.34%	30.63%	29.94%
718	4	38.86%	35.79%	33.74%	32.93%
911	5	33.21%	38.37%	36.05%	35.05%
1104	6	32.06%	40.41%	37.92%	36.77%
1297	7	30.60%	40.41%	39.54%	38.10%
1490	8	28.92%	38.58%	40.64%	39.04%
1683	9	27.17%	32.83%	41.39%	39.61%
1876	10	25.26%	31.60%	40.39%	39.72%
2069	11	23.01%	29.21%	39.90%	39.50%
2262	12	19.97%	25.82%	34.71%	38.09%
2455	13	17.08%	22.76%	28.87%	35.76%
2648	14	13.34%	19.01%	26.38%	32.95%
2841	15	9.09%	14.85%	22.45%	29.75%
3034	16	15.52%	20.97%	27.97%	34.89%

CONCLUSION

- In the project work we found that the results shown by manual computation of design and at shown by software were same. Also the design can be done by two different ways in STAAD PRO. one is RC Designer method and the other is the simple member design. RC Designer method is also called physical member method and interactive design method. The main difference between the two is that in member method each and every member is considered as a single element and hence for each element the design is carried out separately. In physical member method the members which are columns and beams lying in single straight line are designed based on the most critical member and hence the same results are applied to all. Our difference which was found was that physical member method is more detailed than the member method hence useful in situations where failure needs to be understood.
- Assigning slab or replacing it by floor load is one and the same thing but while providing slab the end conditions must be clearly defined.
- It is found that the max. moment exists at the top members. The reason for this is that as we move up the amount of relative deflection increases. There is no bracings provided to prevent the sway. Hence apart from moment added due to loads applied extra moment will be acting due to relative displacement given by.
- FEM due to relative displacement Δ is given as . Hence more Δ means more moment.
- Also it is found that moment will not be max. at at the centre because the end conditions are fixed.
- It has also been observed that the composite built up members are more useful than simple in case of two way bending because they provide nearly equal moment of inertia in both directions whereas I-sections although provide more moment of inertia about the major axis but less about the minor axis and hence not at all useful when bending about minor axis cannot be prevented.
- The axial force coming on a paricular column reduces by providing infills. The possible reason is that when at a joint there are large no. of members connected the load gets distributed. If there would have been no infill the load would have directly transmitted to the next column but now it gets distributed. Hence the load carrying capacity of the particular column increases by providing infills.
- From the table- it is found that the infills reduces the moment coming on a particular element, which states that providing infills increases the overall strength of the structure. Now the reason for reduction in moment at a particular eement is the same that at a particular node more elements ate meeting which are sharing the load which was earlier taken by single element.
- The amount of max. horizontal displacement reduces approx. 2 times when we are providing infillsin the entire storey, which states that the infills increases the overall stiffness of the structure. Since stiffness($k = \frac{P}{\Delta}$) and displacement, Δ for 100% infills is coming less than without infills hence the stiffness increases.

- The amount of displacement is almost 1.50 times more with 25% and nearly the same with 50% infills. Also with 75% infills the amount of max. horizontal displacement reduces by 1.55 times. Hece we can say that if we are providing less than 50% of infills in the structure than we must take into account the effect their effect as the displacement produced is more thereby resulting in more stress at a region and hence causing more possibility of damage. If infills are more than 50% their effect may or may not be taken, but if takn it will reduce the cost of structure.
- In case of 100% infills in the structure the amount of storey drift that is the relative displacement between two adjacent storeys is 63% less than that without infill.
- The amount of lateral force also reduces by infills.

ANNEXURE-A

- **Comparison of floor load by manual computation (2-D) and STAAD.PRO(3-D)**

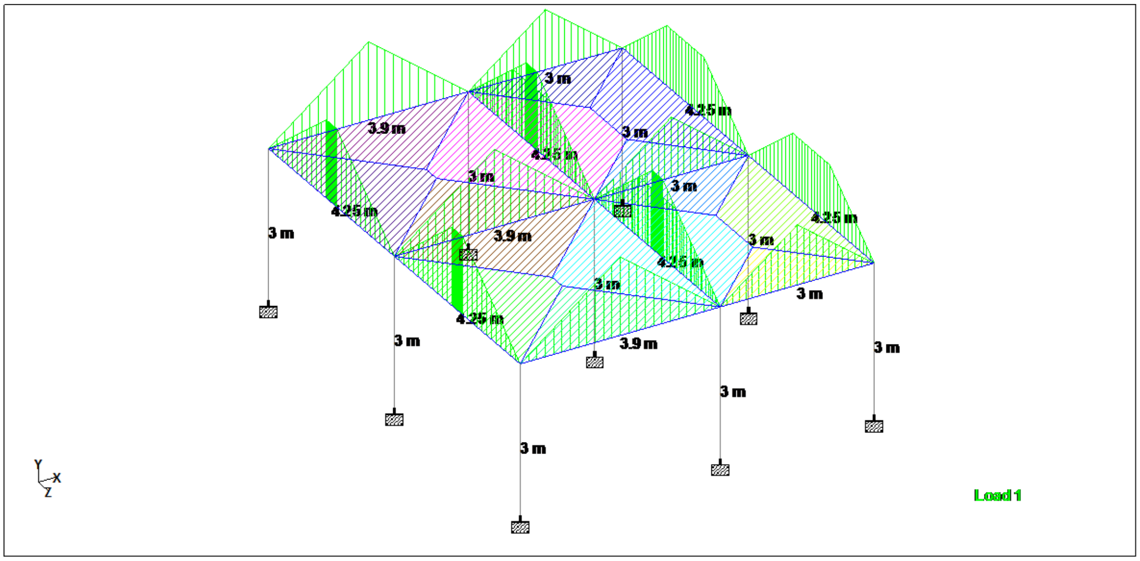


Figure 22: Loading on structure

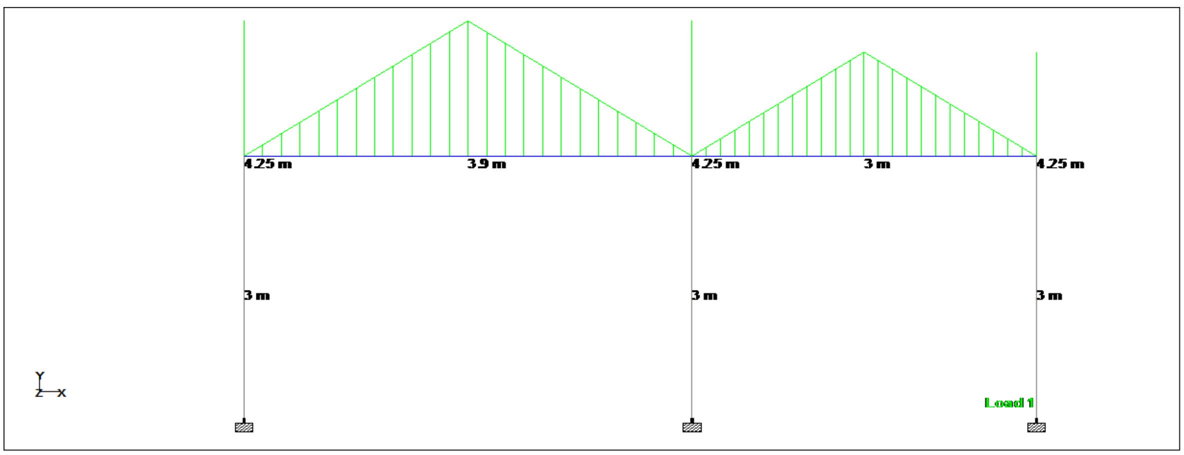


Figure 23: Side view of loading

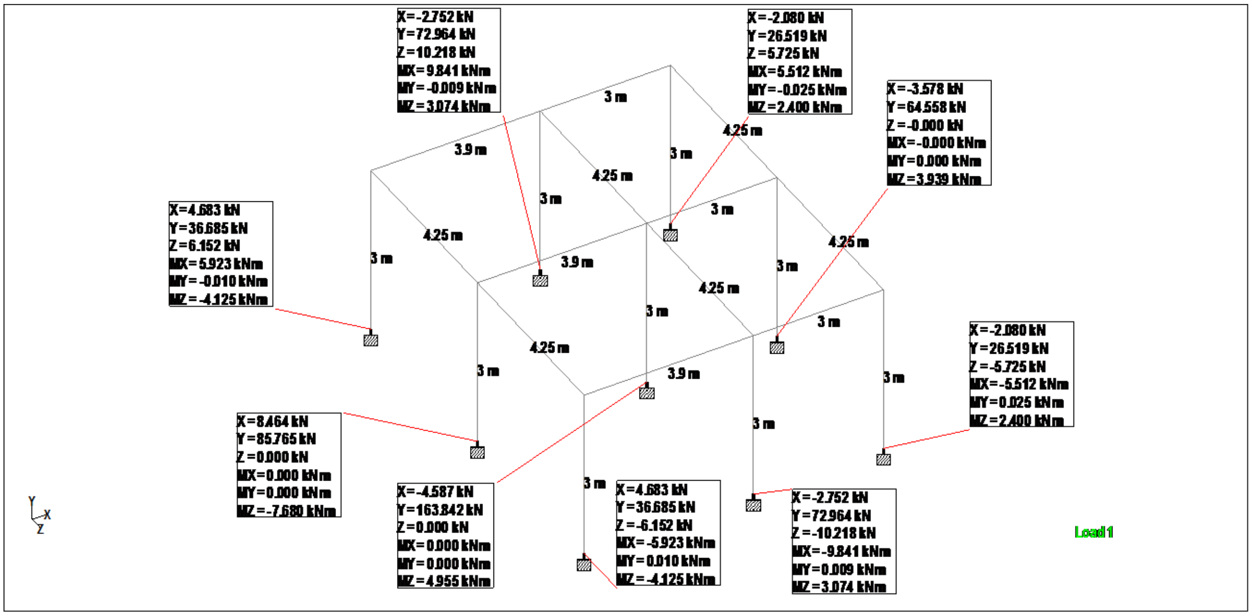


Figure 24: Results

1) Comparison of results between manual calculation and STAAD.PRO

Table 11: Moments

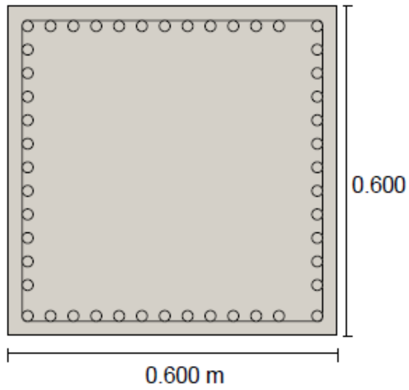
Moment	Manual calculation(KNm)	STAAD.PRO result(KNm)
M_{ab}	-3.9701	-4.125
M_{ba}	-9.0494	
M_{dc}	2.9498	3.074
M_{cd}	4.7904	
M_{ef}	2.1295	2.4

Table 12: Horizontal Reaction

Horizontal reaction	Manual calculation(KN)	STAAD.PRO result(KN)
H_a	4.3398	4.683
H_d	-2.58	-2.752
H_e	-1.7597	-2.080

ANNEXURE-B

- **RCC Beam Software design**



Design Load

Load	8
Location	End 2
Pu(Kns)	6531.870117
Mz(Kns-Mt)	0.020000
My(Kns-Mt)	24.290001

Design Results

Fy(Mpa)	415
Fc(Mpa)	30
As Reqd(mm²)	8984.000000
As (%)	2.681000
Bar Size	16
Bar No	48

Figure 25 : software design of beam

ANNEXURE-C

RCC Column software design

Member 600 - Detailed IS456 Design Requirements
Design of Column is done as per IS:456-2000

Section Property: 600 x 600

Storey height = 3.050 m

Rectangular section: Width = 600 mm Depth = 600 mm

Cover = 40 mm

Member 600 - Detailed IS456 Main Reinforcement

Unsupported Length of column = 3.05 m

Slenderness checks:

Effective length Major, l_{ex} = 3.050 m

Effective length Minor, l_{ey} = 3.050 m

Slenderness ratio Major, (l_{ex}/H) = 5.083

Slenderness ratio Minor, (l_{ey}/B) = 5.083

Slenderness Limit = 12.000

NOT SLENDER

Critical Loadcase: LOADCASE 8

Axial Load P, = 4177.05 kN

Major M, End 1, = -52.21 kNm

End 2, = 52.68 kNm

Minor M, End 1, = 1.34 kNm

End 2, = -1.17 kNm

Minimum eccentricity about major axis = 26.10 mm

Minimum eccentricity about minor axis = 26.10 mm

Moment Due to Min Ecc Major = 109.02 kNm

Moment Due to Min Ecc Minor = 109.02 kNm

Min. Eccentricity moment about major axis ignored as per Cl 39.3

Min. Eccentricity moment about minor axis ignored as per Cl 39.3

Design Moment Major = 52.68 kNm

Design Moment Minor = 1.34 kNm

Steel area required = 3167mm^2 (28 No. 12 dia. bars)

Total steel area provided = 3167mm^2

Pure Axial Capacity P_u =7408.639 KN

Axial Capacity Ratio P/P_u =0.564

Axial Capacity ³ Axial Load OK for axial resistance

Major Axis Capacity M_{ux1} =573.999 KNm

Major Axis Capacity Ratio M_{ux} / M_{ux1} =0.092

Major Axis Capacity ³ Major Axis Moment OK for moment resistance

Minor Axis Capacity M_{uy1} =573.999 KNm

Minor Axis Capacity Ratio M_{uy} / M_{uy1} =0.002

Minor Axis Capacity ³ Minor Axis Moment OK for moment resistance

Biaxial Interaction equation = $[(M_x/M_{ux1})]^{an} + [(M_y/M_{uy1})]^{an}$ 1,0

Cl. 39.6

where exponent, α_n = 1.606

Biaxial Interaction equation = $[(52.68/574.00)]^{1.61} + [(1.34/574.00)]^{1.61}$ = 0.56

Biaxial Interaction Result 1.0 OK for biaxial resistance

<u>Member 600 - IS456 Transverse Reinforcement</u>

Distance between compression bar and a restrained bar in major axis > 150 mm Cl 25.5.3.2

ANNEXURE D

Steel beam Software Design

Design Of Member No. 324 As Per IS:800

STEEL BEAM

Input Parameters

Member Section	ISMB600
Cross Sectional Area A_x (m^2)	0.02
Net section factor	1.00
Net Effective Sectional Area A_{net} (m^2)	0.02
Shear Area Along Major Axis A_z (m^2)	0.01
Shear Area Along Minor Axis A_y (m^2)	0.01
r_z (m)	0.24
r_y (m)	0.04
Section Modulus About Major Axis - Tension Edge S_{tz} (m^3)	0.00
Section Modulus About Major Axis - Compression Edge S_{zz} (m^3)	0.00
Section Modulus About Minor Axis - Tension Edge S_{ty} (m^3)	0.00
Section Modulus About Minor Axis - Compression Edge S_{yy} (m^3)	0.00
Unsupported Length - Major Axis For Slenderness Check L_z (m)	3.90
Unsupported Length - Minor Axis For Slenderness Check L_y (m)	3.90
Effective Length For Allowable Bending Stress Calculation Unl(m)	3.90
Yield Stress f_y (MPa)	250.00
Allowable Ratio For Interaction Check	1.00

Design Forces

Combined Axial Force & Bi-axial moment

Axial Load F_x (kN)	-1.66
Major Axis Moment M_z (kN m)	289.12
Minor Axis Moment M_y (kN m)	0.00
Critical Loadcase No.	8
Critical Section (m)	0.00
Beam No.	2629
Shear Along Major Axis	
Shear Along Major Axis F_z (kN)	2.05
Critical Loadcase No.	23
Critical Section (m)	0.00
Beam No.	2629
Shear Along Minor Axis	
Shear Along Minor Axis F_y (kN)	217.17
Critical Loadcase No.	8

Critical Section (m)		0.00
Beam No.		2629
Details Of Calculation		
Slenderness Checking		
L_{ez} (m)		3.90
L_{ey} (m)		3.90
L_{ez} / r_z		16.11
L_{ey} / r_y		95.39
Actual Slenderness Ratio		95.39
Allowable Slenderness Ratio		400.00
165.00		SAFE

Check Against Axial Tension And Bi-Axial Bending

Bending Compression governs for Cl 7.1.2

$=f_{cz} / f_{cz\_allowable} + f_{cy} / f_{cy\_allowable}$

Bending about Major Axis :

Actual Bending Compressive Stress - Major Axis f_{cz} (MPa)	$=Mz / S_{zz}$	96.10
Allowable Bending Compressive Stress- Major Axis $f_{cz\_allowable}$ (MPa)		125.50

Bending about Minor Axis :

Actual Bending Compressive Stress- Minor Axis f_{cy} (MPa)	$=My / S_{yy}$	0.02
Allowable Bending Compressive Stress- Minor Axis $f_{cy\_allowable}$ (MPa)		$=0.66f_y$

Interaction Checking

Interaction ratio		0.77
Status		SAFE

Check Against Shear

Shear Stress Along Major Axis V_z (MPa)	$=F_z / A_z$	0.36
Shear Stress Along Minor Axis V_y (MPa)	$=F_y / A_y$	30.16
Allowable Shear Stress $V_{\_allowable}$ (MPa)	$=0.4f_y$	100.00
Interaction ratio (Along Major Axis)	$=V_z / V_{\_allowable}$	0.00
Status		SAFE
Interaction ratio (Along Minor Axis)	$=V_y / V_{\_allowable}$	0.30
Status		SAFE

ANNEXURE-E

Steel Column Section Properties

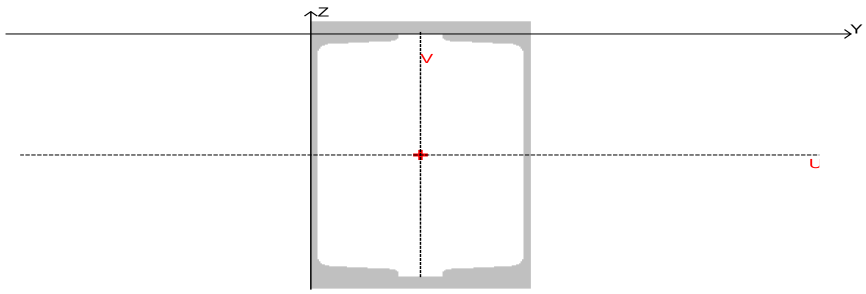


Figure 26: Section details

Table 13 : Compound section

Section element	**Material**	**E (mton/cm^2)**
S Shapes ISMC400	Steel	2090.27
S Shapes ISMC400	Steel	2090.27
Sheet 250 x 20	Steel	2090.27
Sheet 250 x 20	Steel	2090.27

The overall dimensions of the section are 250 x 440 mm

Basic geometry of the section

Table 14 : Compound section properties

	Parameter	Value	
A	Cross sectional area	225.86	cm^2
□	Angle between Y-Z and U-V axes	-0.0	Deg
I_y	Moment of inertia about axis parallel to Y passing through centroid	74298.93	cm^4
I_z	Moment of inertia about axis parallel to Z passing through centroid	18433.29	cm^4
I_t	Torsional moment of inertia (St. Venant)	191.82	cm^4
i_y	Radius of gyration about axis parallel to Y passing through centroid	18.14	Cm

i_z	Radius of gyration about axis parallel to Z passing through centroid	9.03	Cm
W_{u+}	Elastic modulus about U-axis (+ve extreme)	3377.22	cm^3
W_{u-}	Elastic modulus about U-axis (-ve extreme)	3377.22	cm^3
W_{v+}	Elastic modulus about V-axis (+ve extreme)	1474.66	cm^3
W_{v-}	Elastic modulus about V-axis (-ve extreme)	1474.66	cm^3
$W_{pl,u}$	Plastic modulus about axis parallel to U-axis	3864.08	cm^3
$W_{pl,v}$	Plastic modulus about axis parallel to V-axis	1879.58	cm^3
I_u	Moment of inertia about U-axis	74298.93	cm^4
I_v	Moment of inertia about V-axis	18433.29	cm^4
i_u	Radius of gyration about U-axis	18.14	Cm
i_v	Radius of gyration about V-axis	9.03	Cm
a_{u+}	Centroid to edge of compression zone along +ve U-axis	6.53	Cm
a_{u-}	Centroid to edge of compression zone along -ve U-axis	6.53	Cm
a_{v+}	Centroid to edge of compression zone along +ve V-axis	14.95	Cm
a_{v-}	Centroid to edge of compression zone along -ve V-axis	14.95	Cm
y_M	Distance to centroid along Y-axis	12.5	Cm
z_M	Distance to centroid along Z-axis	-20.0	Cm
y_P	Distance to equal area axis along Y-axis	12.5	Cm
z_P	Distance to equal area axis along Z-axis	-20.0	Cm

ANNEXURE-F

Bottom Storey As Soft Story:

Table 15 : Maximum node displacement

		Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	1187	24.648	-17.826	-13.424	33.249	0	0	-0.001
Min X	106	-1.121	-1.532	-0.251	1.915	0	0	0
Max Y	1	0	0	0	0	0	0	0
Min Y	1222	18.608	-24.633	-17.229	35.354	0	0	0
Max Z	137	0.406	-1.447	1.275	1.971	0	0	0
Min Z	1205	14.302	-17.798	-21.426	31.311	-0.001	0	0
Max rX	1160	14.131	-18.077	-6.789	23.928	0	0	0
Min rX	137	0.665	-1.997	-1.812	2.777	-0.001	0	0
Max rY	1157	22.393	-16.766	-12.372	30.588	0	0	0
Min rY	1208	13.704	-16.97	-19.585	29.315	0	0	0
Max rZ	1194	5.702	-17.774	-13.39	22.972	0	0	0
Min rZ	92	1.772	-2.02	-0.524	2.738	0	0	-0.001
Max Rst	1222	18.608	-24.633	-17.229	35.354	0	0	0

Table 16 : Maximum reaction 1

		Horizontal	Vertical	Horizontal	Moment		
	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	99	96.988	6216.249	-7.01	15.545	-0.119	-141.822
Min Fx	100	-68.865	3493.584	-4.279	8.293	0.208	145.131
Max Fy	72	7.029	7764.708	-6.491	21.066	0.288	22.286
Min Fy	127	-49.77	2540.466	-6.268	9.475	0.846	120.394
Max Fz	116	0.923	3453.151	72.675	153.267	0.164	17.514
Min Fz	130	-1.478	5936.125	-107.791	-161.249	0.105	35.427
Max Mx	116	1.581	5872.759	67.793	156.305	0.147	29.399
Min Mx	130	-1.478	5936.125	-107.791	-161.249	0.105	35.427
Max My	132	-12.617	4920.615	43.118	125.84	1.616	50.446
Min My	1	-36.279	4749.921	31.067	70.527	-1.802	111.123
Max Mz	153	-68.388	5666.432	-10.995	15.818	0.222	155.821
Min Mz	99	96.988	6216.249	-7.01	15.545	-0.119	-141.822

Table 17 : Maximum reaction 2

		Horizontal	Vertical	Horizontal	Moment		
	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	6	70.968	5455.85	24.663	6.629	2.118	-149.74
Min Fx	35	-84.838	6310.091	1.346	-20.273	0.09	125.752
Max Fy	141	6.56	7888.932	14.75	-0.118	0.472	-39.447
Min Fy	7	55.867	2574.967	1.925	-16.071	0.205	-125.918
Max Fz	3	9.615	6228.949	98.826	146.757	0.437	-42.799
Min Fz	35	-22.025	5686.199	-70.434	-148.602	0.039	5.303
Max Mx	3	9.615	6228.949	98.826	146.757	0.437	-42.799
Min Mx	21	-17.031	5441.597	-70.308	-150.345	-0.184	-3.084
Max My	99	-23.729	5371.748	-7.546	-34.836	3.318	32.591
Min My	130	-3.407	7037.478	-38.284	-81.059	-2.739	-22.055
Max Mz	91	-84.733	6214.105	-12.69	-46.046	0.418	125.941
Min Mz	4	70.443	5749.115	32.305	19.59	1.22	-150.276

Table 18 : Maximum beam force

	Beam	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	73	72	7764.708	-7.029	-6.491	0.288	-21.066	22.286
Min Fx	4927	1054	-208.858	-12.46	0.038	0.005	0.163	14.602
Max Fy	365	200	-98.96	146.273	0.23	0.067	-0.358	103.956
Min Fy	642	283	-48.751	-146.39	-0.243	-0.038	-0.381	103.998
Max Fz	114	116	3453.151	-0.923	72.675	0.164	-153.267	17.514
Min Fz	127	130	5936.125	1.478	-107.791	0.105	161.249	35.427
Max Mx	344	140	4638.532	23.697	-17.808	1.704	34.753	38.109
Min Mx	263	8	4609.122	-10.881	27.916	-1.814	-45.906	-20.6
Max My	127	130	5936.125	1.478	-107.791	0.105	161.249	35.427
Min My	127	137	5897.319	1.478	-107.791	0.105	-167.513	30.919
Max Mz	206	153	5666.432	68.388	-10.995	0.222	-15.818	155.821
Min Mz	98	99	6216.249	-96.988	-7.01	-0.119	-15.545	-141.822

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